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11 YEARS PERFORMANCE OF 18 BITUMINOUS TEST SECTIONS ON A MAJOR URBAN FREEWAY

Engineering
Materials
Office

11 YEARS PERFORMANCE OF 18 BITUMINOUS TEST SECTIONS ON A MAJOR URBAN FREEWAY

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The opinions expressed in this report are those of the authors and do not necessarily reflect the official views or policies of the Ministry of Transportation of Ontario.

ABSTRACT

This report summarizes the experience of a field trial placed on one of the heaviest trafficked roads in North America: Hwy 401, carrying some 250,000 AADT in 1985.

The trial consisted of 18 different test sections representing a comprehensive range of asphalt surface course mixes. Mix designs ranging from sand mix to high stone content mixes of various materials and compositions were involved. The conditions of the test sections were monitored periodically for eleven years and samples were obtained for laboratory testing and analysis.

While the primary objective of this study was to establish the most effective mix designs for improving the frictional characteristics of the travelled surfaces for urban freeways, other parameters relating to the material properties and performance of the mix designs and their various relationships were also evaluated.

The evaluation of these test sections had lead to formation of policies regarding the use of high quality asphalt mixes on provincial highways.

All the test sections performed better than expected for single course thin overlays on concrete pavement under the heavy traffic prevailed. The open and dense mixes performed equally well.

It was found that, for optimum friction characteristics for high speed roads, a surface must possess sufficient macrotexture for bulk surface water drainage and microtexture for penetrating the remaining thin water film in the contact area. In order to design mixtures with these properties, the coarse aggregate content should be set above 50% and 60% for dense and open friction course mixes respectively. Good quality polish resistant aggregate should be used for both the coarse and fine aggregates.

In conclusion, the analysis confirmed that the Ministry policies set forth previously based on the earlier results and the current practice of using Open Friction Course (OFC) and Dense Friction Course (DFC) for treatment of highways requiring friction surfacings are sound and adequate.

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1. INTRODUCTION

Increased understanding of the mechanism of frictional behaviour of the tire to road surface interface in the late sixties has lead to the search for and development of methods to design asphalt mixes with improved surface friction characteristics. Also, the need to construct and maintain safer and better ride quality road surfaces was created by the coming of high speed transportation made possible by the advances in automobile truck and tire technology since the 50's. The dramatic increase in the volume of cars and trucks, brought about by the increase in the post-war ownership of automobiles and the dependence on road transportation of goods and services had also created the need for higher quality highways.

These developments posed new challenges to highway engineers and material designers in that much higher standards of surface and structural properties were called for in the design of pavements. This was clearly felt on major multi-lane urban commuter freeways such as the ones in metropolitan Toronto--the Hwy 401 Toronto By-Pass.

In Ontario, the catalyst which started off the search for better techniques to improve the highway as well as for a hot mix surfacing system that will produce long wearing and good friction properties was the need to rehabilitate and upgrade of this section of freeway. In response to such demands, an evaluation program to determine the most suitable designs which will provide the desirable driving qualities for freeways under the anticipated heavy traffic conditions was initiated in 1972. In the summer of 1974, 18 different designs of bituminous overlay test sections were constructed on the westbound express lanes of Hwy 401 Toronto By-Pass immediately west of the Allen Expressway.

A previous report by Ryell et al (2) detailed the results obtained up to 1978. Since then, some sections had deteriorated to the degree that complete removal of all the test sections was warranted in 1985. Prior to this work, a final evaluation was carried out on the sections and this report puts together all the pertinent information on the evaluation and performance of the test sections over the eleven years of service.

2. TEST SECTIONS

2.1 OCATION OF TEST SECTIONS

The site selected for the test sections was the westbound core lane between the Allen Expressway and Jane Street on the "Toronto By-Pass".

The Toronto By-Pass of the Hwy 401 was reconstructed between 1963 and 1973 when the original four-lane divided highway was widened to a twelve-lane design. A system incorporating the express (core) lanes to provide for fast moving through traffic and collector lanes to allow local traffic to enter the freeway at interchanges was adopted.

The pavement structure consisted of 230 mm (9 in) of reinforced concrete overlaying 305 mm (12 in) of granular base. The concrete pavement was selected for this highway because it was expected to have long service life under the anticipated heavy traffic conditions. However, by 1972, the original burlap drag and broom textured concrete pavement surface was polished by the heavy traffic and studded tires (banned after 1971), reducing the texture depth and hence the friction value to between 19 - 22 as determined by the ASTM Brake Force Trailer at 100 km/h (60 mph).

The rapid reduction of the surface friction within less than 10 years of construction had effectively and prematurely shortened the serviceability of the pavement. The main and earliest problem to develop was the marked increase in the multi-car wet weather accident rate. Grooving the existing concrete pavements was carried out to improve the frictional properties. This technique, although somewhat effective for a while, created excessive tire noise which was unacceptable to both the driving public and nearby residents. Problems were also encountered with the performance of the joints; transverse cracking began to show up between the joints and spalled areas of varying depth developed at quite a number of joints and cracks.

2.2 TRAFFIC DATA

For the central section of the Toronto By-Pass between Dufferin Street and Keele Street where the test sections were located, the traffic count was 179,550 AADT in 1974 and 250,000 AADT in 1985 (Fig 1). The allowable truck weight and axle loadings were 63.5 tonnes and 10.6 tonnes respectively (Table 1). The percent truck traffic was about 17% for both directions.

The percent traffic split of three westbound core lanes is illustrated in Table 2. The relative proportion of traffic split remained the same during the years from 1975 and 85 with each lane carrying about one third of the traffic. However, the driving lane carried about 65% of the truck traffic as compared to the 32% and 2% for the centre and passing lanes respectively.

2.3 LAYOUT OF TEST SECTIONS

The 18 test sections were each 137 m (448 ft) long and 11.3m (37 ft) wide. Sections 1 to 10 and 17 were 38 mm (1.5 in) thick overlays whereas sections 11 to 16 were only 25 mm (1 in) and section 18 (or marked #1-19 in previous report by Ryell, 2) consisted of two lifts of 38 mm (1.5 in) overlay (Fig 2). Details of the layout is described in reference 2 (Detail No.7).

3. CHARACTERISTICS OF MIX DESIGNS AND MATERIALS

There were two alternatives considered for rehabilitating polished concrete pavements; one was to use thin bituminous overlays and the other was to texture the existing concrete pavement surface with transverse grooves. The reasons for grooving are to create sharp edges for the tire to grip on and to provide channels to facilitate surface water drainage to reduce hydroplaning potential of vehecles. Both of these methods were evaluated; the concrete texturing part of the project was reported elsewhere (2) and therefore is not discussed here. As for the alternative of bituminous overlays, eighteen different trial sections were placed to examine the various mix design factors affecting the frictional properties of road surfaces, e.g.

a) types of aggregate
b) coarse aggregate content
c) fine aggregate content
d) different blends of fine aggregates
e) use of asbestos fiber and mineral filler

3.1 APPROACH TO MIX DESIGN

With the view to examine these factors and by recognizing the belief that a surface with good frictional properties must possess sufficient microtexture or harshness (as assessed by the polished stone value and described in the ASTM D3319) and suitable macrotexture or stone projections (as measured by the ASTM E770), mixtures of varying ratios of coarse to fine aggregate as well as blendings of different aggregate types were selected. The selection of mix designs was also based on experience from previous small pilot projects and review of mixes used by other jurisdictions. The test sections included both dense and open graded types of mixtures with a variety of coarse and fine aggregate types including traprock, steel slag and blast furnace slag (Table 3). The design concept of the mastic mix was also included in the trial.

3.2 MIX TYPE AND COMPOSITION

Test sections 1 to 10 (and 18) consisted of HL-1 mixes in which the coarse aggregate content was progressively increased to obtain a greater density of stone particles at the surface. The HL-1 mix is a standard dense graded surface course mix used on main highways in Ontario, consisting either crushed traprock or slag coarse aggregate and locally available fine aggregates. However, different types of fine aggregate ranging from traprock screenings to natural sand were used in these ten sections.

Mixes in sections 11 and 12 were sand asphalt mixes using traprock screenings as fine aggregate. Both mixes contained small percentage of oversize, in the form of 6 mm traprock chips, and asbestos fiber filler (2%).

Section 13 and 14 consisted of open graded mixes designed to have high permeability characteristics that facilitate rapid drainage of surface water into, and laterally through, the surface course layer. The mixes used a large proportion (67%) of single size coarse aggregate (9.5 - 4.75 mm) and a small amount of washed fine aggregate. Mixes in sections 15 and 16 had only 30% coarse aggregate, and the same washed fine aggregate as mixes 13 and 14.

Section 17 consisted a mix called "Mastiphalt" which is a kind of mastic asphalt derived from the German "Gussasphalt" technology. The modification was made so that the material could be mixed and laid with conventional equipment.

The gradations of different mixes are shown in Appendix 1.

3.3 MATERIALS

(a) *Aggregates*

The coarse aggregates used were high quality aggregates possessing good frictional properties as evaluated by the polished stone value (PSV) test. The fine aggregates were either a combination of natural sand and limestone screenings or screenings from the same source as the coarse aggregates. The following describes the materials used in the test mixes.

(i) Traprock: all but four mixes used traprock in whole or in part of the mix. Traprock is a fine-grained very hard basaltic material from the Havelock and Marmora areas. It has a PSV and an aggregate abrasion value (AAV) of about 43 and 3 respectively.

(ii) Steel Slag: The steel slag aggregate was from Stelco of Hamilton and has a PSV of 58 and an AAV of 3.

(iii) Blast Furnace Slag: BFS was supplied by National Slag of Burlington and has a PSV of 54 and an AAV of 15. The high AAV tends to suggest that the BFS is a more friable material than is otherwise desirable for a road surfacing material.

(iv) Sand and screenings: Natural sand from Harris and limestone screenings from Uhthoff were used as fine aggregate in seven of the mixes.

(v) Mineral filler: The crushed fine limestone filler was from Domtar (-30 grade).

(vi) Asbestos: 2% by mass of short asbestos fibre supplied by Johns Manville (7-M grade) was added to mixes placed in six of the sections.

(vii) Traprock screenings: washed screenings were used in the open graded mixes and unwashed screenings in dense graded mixes.

The bulk relative density data of different aggregates were not available from past records. The following values are commonly used for these types of aggregate:

Material	BRD
- traprock	3.21
- traprock screenings	3.19
- steel slag	3.42
- steel slag screenings	3.48
- blast furnace slag	2.58
- blast furnace slag screenings	2.93
- natural sand	2.37
- limestone screenings	2.68
- limestone mineral filler	2.70
- short asbestos	2.69

(b) *Asphalt Cement*

The asphalt cement (AC) used in all the mixes, except mix 17, was 85/100 penetration grade asphalt from Shell, Montreal. The AC for mix 17 was 60/70 penetration grade asphalt supplied by Imperial Oil also from Montreal.

4.0 CONSTRUCTION

The contract for the project was awarded to Warren Bithulithic Ltd. The mixing plant was a Madsen 3630 kg (8000 lbs.) batch plant, situated about 40 km (25 miles) southwest of the test sections. The construction commenced on 74-07-22 and was completed a month later. Night paving only was permitted due to heavy traffic (1).

4.1 MIX PRODUCTION

The production of the trial mixes is described in detail in reference 1. The plant would produce one or two batches of the desired mix and samples were taken for testing for compliance with the mix design requirements and for making any necessary adjustments to the blend or batch weight. The asbestos and/or mineral filler was added by using a vertical bucket elevator and belt conveyor to the weigh hopper. A normal mixing time (40 sec) was extended for those mixes with asbestos due to lumps appearing in the mixes (e.g. for mix #17 which contained 9% mineral filler and 2% asbestos, a total mixing time of 70 seconds was found necessary).

4.2 TRAFFIC CONTROL AND LIGHTING

The core lanes at the contract location was closed and traffic diverted via a transfer lane in time for the contractor to commence work at 8 pm each night Monday through Thursday. Due to night paving, in addition to the highway illumination, a lighting system consisting of one telescopic tower with double flood-light was used. Also, there was a truck with generator and four spotlights which accompanied the spreader.

4.3 PLACEMENT OF MIXES

The asphalt overlays were placed using a conventional Cedarapids tracked paver and compacted with a 9072 kg steel-wheeled breakdown roller, a 8165 kg rubber-tired intermediate roller and a 7258 kg steel-wheeled finishing roller. The shoulders were paved with a Pavemaster driveway spreader due to the confined working space on the shoulder.

The sequence of operations of each night involved first sawcutting and removing the temporary asphalt ramp placed at the completion of the previous night's work. This was quickly followed by brooming and the

spraying of SS-1 emulsion tack coat for the next 137 m (448 ft) section. After allowing for 5 to 15 minutes of curing of the tack coat, paving of the asphalt began and took about two hours to complete.

For compacting the mixes with asbestos, it was found necessary to stop using the rubber-tired roller because of the excessive pick-up of the mix on the tires. This problem was not experienced with the other mixes. The weather was good and the paving was completed in sixteen nights of work with only one night of cancellation due to rain.

4.4 JOINT TREATMENT

In order to reduce any random cracking in the overlay, joints were sawn and sealed directly over the joints in the concrete pavement. The sawing was done by a self-propelled concrete saw to a width of 20 mm (3/4") the next night after the section was paved.

Treatment to the transverse and longitudinal joints by sawing and filling of grooves over joints in the concrete pavement is detailed in reference 1. For the 25 mm thick sections, 19 mm wide grooves were cut to the full thickness and filled with hot poured rubberised asphalt sealer. For 38 mm and double 38 mm lift thick sections, grooves were cut to 25 mm depth only and filled with 13 mm of cane fibre expansion joint filler at the bottom, and filled with hot poured rubberised asphalt sealer. However, no treatment was made for sections 3 and 16 and half the length of section 18.

5. INITIAL EVALUATION AND POLICY ON FRICTION COURSE

Ryell et al (2) reported the initial evaluation results obtained up to 1978. The report describes a full range of tests programmed to monitor the frictional characteristics of the various test sections. Other properties relating to the riding quality and performance of the test sections were also discussed.

5.1 REPORT FINDINGS

The findings presented in that report suggested that in order for a surface to produce good levels of friction at high speed, it must have sufficient macrotexture for drainage of bulk water from the tire to road contact area, and microtexture to penetrate the remaining thin water film

to promote contact between the tire treads and road surface.

Substantially better friction was obtained from bituminous mixes than polished concrete. All the mixes had a decline in friction during the first three years of service due to reduction in macrotexture as coarse aggregates were pressed into the matrix. The degree of decline is influenced by the mix design as well as traffic. The friction reduction was most pronounced in the driving lane where truck traffic prevails.

The mixtures that provide good friction value were :-

(i) dense graded mixes with both coarse and fine aggregates consisting of traprock, steel slag or blast furnace slag,

(ii) open graded mixes with high stone content using traprock coarse and fine aggregates.

Mixes with the lowest friction values were those containing fine aggregates consisting of natural sand blended with limestone screenings. The dense grade mixes containing traprock or slag screenings had attained better macrotexture than mixes containing sand. Also, dense mixes using steel slag and blast furnace slag aggregates provided better friction values than the traprock aggregate used.

5.2 POLICY ON USE OF FRICTION COURSE MIXES

Based on the performance of these test mixes, a Ministry policy was introduced in 1978 governing the selection of surface course mixes for main highway facilities. The policy specified the use of Open Friction Course (OFC) mixes (as in sections 13 and 14) as the surface layer for urban freeways and Dense Friction Course (DFC) mixes (similar to sections 3, 7 and 9) for other heavily trafficked main highways having traffic of greater than 5000 AADT.

6. PERIODIC MONITORING AND FINAL EVALUATION

As the main thrust of the project was to develop surface course mixes which will provide adequate and long lasting friction properties for heavy traffic sites, monitoring efforts had been devoted primarily to obtaining comprehensive data on the frictional characteristics of the test sections. However, as the final evaluation program was being drawn up, it was clear that additional valuable data on material performance should be derived from this excercise. Therefore, additional material testing was included in the evaluation and extra samples were taken from the test sections.

6.1 FIELD EVALUATION

In addition to the initial evaluation performed during the first three years, the sections were monitored at different stages of the service life up to the end of the eleventh year in 1985 when the condition of some sections had deteriorated to the stage where complete rehabilitation of the whole test area was necessary. Traffic control was a major undertaking for full testing of all the sections across the three lanes. Hence, evaluation of in-between years was selective and was on certain sections only, notably on sections 1, 3, 7, 9, and 13. However, friction testing by the ASTM E274 Brake Force Trailer was performed on the driving lane in most of the years in the spring and/or the fall.

Field testing include water permeability (Johns Manville method), rut depth measurement and taking stereo photographs for textural analysis by photointerpretation method. As the water permeability test is a field test, it was done only on the mixes that were giving rapid permeability readings. As the mixes were compacted under traffic and voids filled with debris, fewer sections were permeable enough for testing for water permeability in the field.Therefore, cores were taken and tested in the laboratory using an air permeameter.

Rutting measurement was made on some sections on the driving lane after the seventh year when rutting was visible in section 2. However, for the final evaluation, complete crack mapping and condition survey were done on all the sections across the three lanes.

6.2 LABORATORY TESTING

Cores were taken periodically from the driving lane for laboratory testing. For the final evaluation, fifteen cores were taken from each of the test sections and across the three lanes with a total number of 270 cores from the 18 sections. For each section, five cores were taken from each of the three lanes i.e. two from each of the wheel tracks and one between the tracks.

There were different laboratory tests employed and all of the tests were either standard laboratory methods or common ASTM tests.The type of field and laboratory tests performed on the different years are listed in Table 4 and are basically under three catagories:-

(a) *Mixture Analysis*
- Marshall properties of recompacted mixture
- gradation

- resilient modulus (MR): ASTM D-4123
- indirect tensile strength (S_t): ASTM D-4123
- Gyratory Testing Machine (GTM) parameters: ASTM D-3387

(b) *Pavement Property*
- pavement compaction (%)
- pavement voids (%)
- sand patch texture depth
- air permeability: ASTM D-3637
- photointerpretation : ASTM E-770

(c) *Recovered Asphalt Cement*
- penetration (at 25^{o}C)
- viscosity (at 135^{o}C)
- softening point (ring and ball)

7. RESULTS

There are numerous results obtained from the monitoring of these 18 test sections throughout the eleven years. The results are discussed and presented under the following main headings:-

- pavement performance
- surface friction characteristics and various relationship with mix composition
- material properties
- summary of findings

7.1 PAVEMENT PERFORMANCE

In general, the pavement performed well at the early age of the trial. However, since 1982, some local repair was required on a few of the sections, especially sections 9 and 14. The deterioration of these two sections was very rapid and within a period of about three years (i.e. by 1985), complete rehabilitation of the sections had to be carried out. Since it was not practical or economical to resurface only the two sections, the whole 18 sections had to be rehabilitated in 1985. The serviceability of most of the sections was still considered acceptable at that time.

A complete condition survey of the sections was carried out just prior to the rehabilitation work and the results are plotted on the maps as shown in Appendix 2. During the survey, observations were made on the

type of distresses and their severity and frequency of occurence. Weigted factors were then given to the severity and frequency of the distresses based on the principles outlined in the "Manual for Condition Rating of Flexible Pavements" (7). The weighted distresses were totaled up to form the overall performance rating (Table 5 and Fig.3). A high rating indicates poor performance and vice versa.

The distresses observed during the survey are:-

- transverse and longitudinal cracking
- ravelling of surface aggregates
- delamination
- joints and deteriorated areas
- rutting in the wheel tracks.

(a) *Transverse and Longitudinal Cracking*

Cracks were observed on both sides of the transverse joints that were formed over the joints of the underlying concrete pavement. The patterns were similar to the edge cracking seen along the shoulder joints. There were cracks that were caused by saw cuts not directly over the concrete joints as well as intermediate cracks reflected through from the concrete slabs (Fig.4). For the open mixes, there were small areas of material lost in the joints and cracks. High crack counts were recorded for sections 5, 11, 12, 16, and 17 (Table 5), but the results do not suggest any difference in crack generation between dense and open types of mixes. This is because reflection cracking is a function of the support conditions of the underlying concrete slabs. However, the five mixes which contained 2% of asbestos appeared to have higher transverse cracking rating (see App. II). This tends to suggest that the cracks were related to the use of asbestos. The asbestos could soak up the binder and reduce the 'free' binder available for coating the aggregate, thus reducing the film thickness, stiffening the mixes and reducing the flexibility.

In reviewing the crack maps on transverse and longitudinal joint repairs (Table 6), sections 3 and 6 appeared to have the highest total length of joint repaired. These two sections which contain mixes using traprock coarse and fine aggregates are similar to what it is now called "dense friction course (DFC)". It seems that the change of coarse aggregate content from 60% to 45% (viz. sections 6 and 3 respectively) could have slightly reduced the length of joint repairs, but the use of asbestos in section 6 tends to confuse the issue. It seems also that the use of different types of fine aggregate e.g. natural sand and limestone screenings for mixes in sections 5 and 6 could have the effect on reducing the length of joint repairs from 111.1 m to 75.4 m. However, the results

did not suggest that the open friction course (OFC) type of mixes (sections 13 to 16) required any more joint repairs than the DFCs or the dense graded mixes, although the OFC contained a higher stone content. In effect, Fig. 7a reveals that by increasing the coarse aggregate content to as high as 65% the cracking potential had been reduced to almost zero.

The above discussion shows that the key factor to reduce joint repairs is not by reducing the stone content but to ensure that there is sufficient binder film thickness round the aggregates, as in the case of the OFC mixes.

In respect to the question of longitudinal joint sealing, or the lack of it, the results (Table 6) are not conclusive because repairs had been made periodically to the shoulder and joints as they became necessary.

(b) *Ravelling*

The highest ravelling rating of 18 is obtained for sections 3, 5, 9, 14, and 17 (Figs. 5 and 6). Although there is a slight tendency of more ravelling with high stone content mixes (Fig. 7b), mixes with similar stone content (e.g. mixes 4 and 13) yielded ratings of 3 or less. This could be due to the thicker binder film in these two mixes. On the other hand, the ravelling of the mixes mentioned above is a result of the poor cohesion caused by either low A.C. content (e.g. section 3) or the reduction of effective binder content with the use of absorptive aggregates (e.g. blast furnace slag in section 9) and asbestos (e.g. in section 5,14 and 17). Ravelling of most of the sections occurred along the longitudinal joints except sections 3 and 9 where ravelling was observed randomly across the three lanes.

(c) *Delamination*

Quite extensive delamination was observed in section 9 containing the blast furnace slag aggregate. This mix looked dry and ravelled in places to the extent that water was allowed to penetrate into the surface course and the pavement interface. The delamination in section 14 (Fig. 8) was patched up as shown in App. 2 and again, it could be due to the problem associated with the use of asbestos as discussed before. In both of these sections, the delamination occurred mainly in the driving lane. Since these sections were thin overlays, the worse-than-average bonding between the concrete pavement and overlays had lead to the poor durability and performance observed. The construction staff recalled that tack coating on the concrete pavement was 'spotting' and 'scanty'. This does not promote good interfacial bonding. Additionally, as a dense

shoulder mix (HL-3) was used next to the open mixes (e.g. section 9 and 14), it created a water drainage problem in that the water, which drained through the open mixes to the shoulder, was held up by the dense mix and keeping the open mixes moist for a longer period of time. This undermined the bonding between the overlay and the concrete surface by stripping as a result of prolonged soaking in wet conditions. Signs of stripping were indeed observed on cores taken from the test sections. Also, open mixtures tend to retain more water and keep the pavement wet longer than dense mixes. Hence more delamination was observed in the mixes with higher stone content as shown by the trend in Fig. 7c.

Another aspect which affected the delamination of the mixes along the joints in the driving lane was pumping. Pumping was observed on some of the joints where water was continuously pumped to the surface by moving traffic on the unstable concrete slabs in wet weather. This will again provide the moisture conditions that encourage stripping and debonding.

One observation which highlights the importance of sufficient binder film thickness on the performance of pavements is by comparing the high delamination rating of section 14 with the lower value of section 13. The two sections consisted of mixes with the same material and composition. The only difference was that section 14 contained 2% of asbestos. The two sections were also next to each other and therefore subjected to the same environment and service conditions. Yet section 14 had more delamination. This could be explained by the fact that the use of asbestos in the mix might have soaked up the asphalt cement and thus reduced the adhesion of the overlay to the concrete pavement.

(d) *Rutting*

Medium rutting was observed on section 2 (Fig. 9). An average rut depth of 15 mm (Table 7) was recorded on the heavily trafficked driving lane. Insignificant amount of rutting was measured on sections 8, 9, 10, and 18 on both the driving and centre lanes. However, the rutting on sections 9 and 10 was due to wear and plugging of aggregates along the wheel tracks and not because of the deformation of mixtures as in the case of section 2.

The important points which came out of these test sections on rutting performance are that:-

(i) the marginal standard mix (e.g. HL-1, section 2) is not stable enough for the type of traffic on the Hwy 401,

(ii) the sand mix, consisting of all crushed aggregate, did not rut under the same traffic. This is because of the high particle interlocking properties produced by the crushed aggregates in the mix (sections 11 and 12),

(iii) asphalt mixes can be designed to withstand the heaviest traffic experienced on Ontario highways, and

(iv) proper gradation and aggregate selection is the key to solving the rutting problem.

There was virtually no rutting observed in the other sections or in the centre and passing lanes. However, by comparing sections 1 and 2, a replacement of a small percentage of fine aggregate, e.g. 14% of the limestone screenings by traprock screenings did not improve the stability property of the mix in section 2. In effect, the use of traprock screenings had reduced the V.M.A. and voids in the mix (Table 3, field results) which caused the rutting. The small percentage of traprock was not sufficient to provide the firm particle interlock between the crushed aggregate that was separated by the natural sand.

(e) *General*

In general, it appears that for good durability with minimum patching and good resistance to ravelling and delamination, overlays on concrete pavement should consist of a dense mix or at least a dense binder course if an open mix is used as demonstrated by the performance of section 18. The excellent overall rating of this mix can be said to have been attributed to the use of a dense binder course which produced a good bond between the overlay and the concrete pavement. Also the performance results show that existing concrete surfaces must be properly tack coated before resurfacing. The use of asbestos did not seem to contribute to good performance. However, irrespective of mix designs, observations from these test sections confirm the belief that good binder film thickness results in good durability.

However, the limited results relating the coarse aggregate content (% C.A.) with the general performance of the asphalt mixes (Fig. 7d) suggest that mixes with % C.A. between 35 to 45 turned out to be the worst. Increased or decreased % C.A. above and below such range tend to improve the performance.

7.2 FRICTION CHARACTERISTICS

The friction characteristics of the pavement were evaluated mainly by the ASTM brake force trailer and the photointerpretation method (ASTM E-770). Other methods such as British Pendulum Tester and sand patch texture depth (T.D.) were also used at the 7th and 11th years respectively. The schedule of friction and other testing is shown in Table 4. The following describes the different friction properties and compares the performance of the mixes.

(a) *Development of Friction with Time*

Table 8 lists all the friction numbers (FN) at 50 and l00 km/h for the driving, centre and passing lanes at different monitoring periods. The results are also plotted in Figs. 10 to 15. An equilibrium FN (EFN, Table 9) is used to evaluate the different mixes and is obtained by averaging the FN values beyond the third year, i.e. 1977. This is because all the mixes suffered a sharp decrease in FN within the first 2 to 4 years which then fluctuated from year to year depending on the season or the month at which the test was performed. It appears that mixes with high initial friction values took longer to arrive at their equilibrium (e.g. sections 7 versus 1).

(i) Driving Lanes

After 11 years of service, an average equilibrium FN100 of above 31 was obtained for the good mixes (sections 7, 9, 13, 14 in Fig. 16). For the medium mixes (sections 3, 6, 11, 12, and 16) and poor mixes (sections 1, 2, 4, 5, 8, 10, 15, 17 and 18) an average EFN value of 29 and 25 was obtained respectively. The following explains the differences:

- The good sections (7 and 9) were DFC mixes consisting of all-slag aggregates..

- The reasonably high EFN 50 obtained for sections ll and l2 is due to, firstly, the use of all-crushed traprock aggregate and secondly, the close surface texture that provides the high contact area with the tire at low speeds, e.g. at 50 km/h, where surface water drainage is not a problem. However, the rapid drop in FN at higher speeds (i.e. 100 km/h) shows that there was inadequate macrotexture.

- Medium EFN100 mixes were those using all-traprock aggregate and having close macrotexture due to the embedment and flattening of stone particles on the surface.

- Mixes consisted of fine aggregates with natural sand and limestone screenings produced low EFN100 results. It seems that the 12% limestone screenings in the mix of section 8 had reduced the EFN100 to 27 as compared to a value of 34 obtained for section 7 containing a mix with only steel slag fine aggregate. The low EFN (at both the 50 and 100km/h) obtained for section 17 is a result of the high A.C. content and the use of mineral filler and asbestos in the mastic mix.

(ii) Centre Lane

The EFNs obtained in the center lane at both test speeds were higher than the values for the driving lane. The ranking of the EFN of the mixes is similar to the driving lane. The effect of macrotexture in improving the EFN was observable when comparing the results of sections 11 and 12 with sections 13 and 14 at the test speed of l00 km/h. The more open texture mixes in sections 13 and 14 produced higher friction values.

(iii) Passing Lane

With much less traffic travelling on the passing lane, the reduction in both the macro and microtextures was less. Hence, the EFNs of all the mixes were very high for both speeds tested. Again the relative ranking of the mixes is similar to the other two lanes.

(b) *Mix Type and Friction Number*

Fig. 17 shows clearly the ranking of friction performance with time of different mix types at 100 km/h. The steel slag dense friction course (DFC) (section 7, Fig. 18) consistently maintained the highest FN throughout the pavement life. It was then followed by section 13 (Fig.19), an open friction course (OFC) using traprock, and section 3 (a DFC using traprock aggregate). The standard HL-1 mix reached its equilibrium FN within the first year while the other mixes (e.g. sections 3, 7 and 13) took about three years. The HL-1 mix had the lowest FN among the test sections.

(c) *Friction Number versus Speed*

The friction number and speed relationship is illustrated in Figs. 20 to 22. The FN and speed gradient is commonly used to evaluate the textural properties of a surface with change in speed. The mixes were analysed in groups of having (a) increasing coarse aggregate (C.A.) content (b) similar coarse to fine aggregate ratio and (c) natural sand as a fine aggregate.

The FN/speed gradient seemed to be largely affected by the C.A. content rather than by the type of fine aggregate used. Progressive increase in % C.A. tended to increase the macrotexture and reduce the gradient of the FN/speed curve as illustrated in Figs. 20 for the mixes in sections 11, 15, 3, 6 and 13. The other mixes which clearly showed up the differences in the gradient as a result of the change in texture were sections 12 and 14 which consisted of 9% and 67% of C.A. respectively.

The benefit of the coarse macrotexture of sections 13 and 14 (OFC mixes) in facilitating drainage of surface water was realized when relative high EFN100 was maintained at test speed of 100 km/h relative to the values at 50 km/h

Mixes with similar coarse to fine aggregate ratios e.g. sections 1, 3 and 7 tended to produce similar gradients (Fig. 20). However, the level of FN at any speed under consideration is also a function of the quality of the aggregate used.

The FN to speed relationship of the mixes in the centre and passing lanes was relatively the same except that the gradients were more gradual and at a higher level of FN. This is due to the reduced compaction and polishing of the surface textures by traffic (Figs. 20, 21 and 22).

(d) *Sand Patch Texture Depth*

Due to factors such as time and traffic control, the sand patch texture depth (T.D.) was measured in the laboratory on the six-inch diameter cores from the field. The results (Table 10) showed that the wheel track T.D. on average was lower for the driving lane (average = 0.46 mm) than for the centre lane (0.55 mm) or the passing lane (0.62 mm). This again is due to less traffic compaction. The T.D. in between the W.T. was lower than that for the W.T. because of the closing of textures by the softening effect of oil spills from the engines on the asphalt surfaces.

By comparing sections 11, 3 and 6, the wheel track T.D (e.g. 0.28, 0.67 and 0.74 mm respectively) at the driving lane showed that an increase in C.A. content had generally resulted in an increase in T.D. of a mix .

(e) *Photointerpretation Parameters*

Photographs (Figs. 18,19 and 23 to 27) were taken in the wheel tracks

of each test section immediately after construction and subsequent periods as shown in Table 11. The photointerpretation technique was used to study the textural parameter changes which occurred under traffic (2). However, due to some differences in interpretation, some of the values obtained for the final monitoring year (1985) were actually higher than the originals. As the values were used for comparing the different mixes, it was considered not a problem (see Table 11). The following examines the relationship of the parameters with factors such as FN 100 and texture depth (T.D.) as measured by the sand patch method:-

(i) Parameter 'A' (photointerpretation texture height) and sand patch texture depth (T.D.):

There seemed to be a linear relationship between the two factors at T.D. below 0.8 mm (Fig. 28). Above that, the surface voids which are accessible by the sand may not be detectable by the photo-interpretation method. The correlation coefficient was 0.85 and the equation was $TD = 0.16 + 0.377A$

(ii) Parameter 'A' and FN 100:

Fig. 29 shows that when the values obtained for the all-slag mixes (sections 7 and 9) were excluded in the analysis, greater friction values were obtained by the increase in parameter 'A'. (The effects of the increase in texture depth as relating to proportion of coarse aggregates will be discussed later on in (g).

(iii) Parameter 'D' (photointerpretation texture density) and FN 100:

In Fig. 30, when the all-slag mixes were excluded in the plot, a slight trend was obtained between FN 100 and parameter D. An increase in FN 100 was seen with an increase in the D values for all the three lanes.

(iv) Relationship between calculated and measured FN 100 values:

The relationship was examined with the FN 100 calculated using the formula developed by Holt and Musgrove (6). Previous report by Ryell stated that a correlation coefficient (r) of 0.918 was obtained. However, the correlation as shown in Fig. 31 (and Table 13) gave a (r) value of only 0.53, i.e. relationship is insignificant.

(f) *British Pendulum Test*

The British pendulum test (BPN) was performed at the 7th year of the monitoring period (Table 12). Since the pendulum test simulates slow speed skidding, the values can be regarded as of limited use. This is

because the effect of water drainage on friction at high speeds will not be detected by the small rubber pad on the pendulum. The higher value of sections 11 and 12 on the RWT (BPN = 58) as compared to sections 13 and 14 (BPN = 47) confirms the slow speed sliding conditions where the macrotexture is of secondary importance to the microtexture. However, the good microtexture of the slag mixes which produced a high BPN of 58 in sections 7 and 9 was confirmed by this test.

(g) *Mix Composition and Friction*

Mix composition is considered to be one of the most significant factors affecting the frictional characteristics of a mixture. In particular, the coarse aggregate (C.A.) content has direct influence on the availability and durability of the macrotexture. This is shown in the following analysis of the different friction parameters such as the differences in FN with increased speed of travel, photointerpretation texture height (A) and density (D) etc:-

(i) Relationship of % C.A. and the difference in FN at 50 and 100 km/h:

The graph (Fig. 32) reveals that a clear trend existed i.e. with an increase in % C.A., the drop in FN with increase in speed is reduced. This relationship was maintained across the three lanes and appeared to be nonlinear. The trend was confirmed by looking at the all-traprock mixes (e.g. sections 11, 15, 6 and 13 with increasing C.A. content of 14%, 30%, 60% and 67% respectively) An improvement of about 3 FN units over the 50 km/h can be made by increasing the C.A. content from 0% to 45%. But more significantly, the same improvement was obtained by increasing the % C.A. from 45 to 55% i.e. a drop from a difference of about 12 to 9. The optimum C.A. content for maximum friction characteristics seems to be at about 63%. This is close to the C.A. content used in the present specification for the OFC. In other word, this finding suggests that mix designs for high speed traffic roads should have a coarse aggregate content of close to 63%. However, friction results at 100 km/h (Fig.16) show that, with the same C.A. content, the slag mix (section 7) had higher FN than the traprock mix (section 3), although the friction/speed gradient results (Fig. 20) showed similar drop in FN with speed between 50 and 100 km/h..

A similar trend of increase in the difference between the FN100 and FN50 with decrease in the C.A. content was observed for the centre and the passing lanes.

(ii) % C.A. and parameter A (texture height, mm):

Fig. 33 shows that the maximum A value of 1.3 mm was obtained, again, at a C.A. content of 63% (in the driving lane). The decrease in the A values between the passing lane and the driving lane, as indicated by the shifting of the peak of the curve from the right to left, showed that the macrotexture had closed up by traffics in the D.L.. However, the maximum % C.A. was still maintained at about 63%.

(iii) % C.A. and parameter D (texture density):

Fig. 34 shows that the maximum texture density (65%) occurred also at the point of about 63% C.A. level in the driving lane. Again, similar to the parameter A, the D value decreased with increased trafficking of the road surface. This is shown by comparing the positions of the optimums of the curves for the passing lane and centre lane with the driving lane (i.e. a shift of the peak from the right to the left of the graphs).

7.3 MATERIAL PROPERTIES

The material properties were examined under three areas, i.e. pavement characteristics, asphalt cement (A.C.) properties and strength characterisitcs. As samples were taken and tested only at the eleventh year (in 1985) and the A.C. properties and material strength data were not available from the original materials at construction, the analysis on the change of material properties with time is not possible. The following describes the results obtained:-

(a) *Pavement Voids*

In all cases, the pavement void content in the driving lane (D.L.) was lower than the centre and passing lanes due to more traffic compaction (Table 14). Section 2 had a very low pavement void content (average of three lanes = 2.8%) leading to the rutting in the driving lane as discussed previously. The dense friction course mix (section 3) had voids close to or slightly higher than the open friction course in section 13 and 14. Therefore, the OFC void content was unusually low in this case (about 9% in the D.L. and 11% in the P.L.) as compared to those obtained (about 18%) from mixes in current use. Even though there was less traffic on the passing lane, the voids were plugged up with dirt and sand. As observed from the core samples from the driving lane W.T., the surface texture of the DFC mixes (sections 3 and 6) was closed up by traffic, preventing the debris from getting into the open matrix and keeping the pavement voids

high. This explains why high % voids was recorded for some of the cores from the dense mixes (Table 14). The all-slag mixes (sections 7 and 9) had the highest void content among the test mixes.

The relevance of void content in affecting the durability of asphalt mixtures was examined in respect to the permeability and aging of A.C. properties. These two properties are closely related since higher voids allow more accessibility to air and water and hence more rapid aging of the A.C. This two factors are discussed below.

(b) *Permeability*

As a requirement, open graded mixes (e.g. OFC) are designed to have high permeability for the purpose of rapid surface water drainage to improve friction and reducing splash and spray. As the surface water is allowed to drain through the mix, the possibility of hydroplaning at high speed is reduced.

Permeability tests were performed initially on the driving lane wheel tracks of each of the test sections using the Johns Manville outflow permeameter (described in reference 2). However, at the eleventh year, the wheel track surface texture of all the sections was closed up to the extent that it required a much longer time (> 1 hr) to obtain a water permeability reading. Hence, cores were taken from the test sections and tested in the laboratory for permeability using the air permeability method.

The results (Table 15) indicated that the open graded mixes (e.g. OFC as in sections 13 and 14) had a very high initial permeability but the voids closed up after 7 years of traffic compaction, reducing the permeability to about 10 ml/min. For the dense mixes (e.g. DFC as in sections 3 and 9) the values were reduced to zero within the same period (Fig. 35). That means that the effectiveness of the OFC in draining water from the surface is reduced to a level similar to a new DFC after two years of trafficking.

However, the air permeability of the OFC at the eleventh year on the driving lane is still relatively higher than the other mixes, e.g. at 10^{-8} cm versus < 10^{-9} cm respectively. In general, the lower air permeability in the driving lane confirms the belief that the traffic compaction in the driving lane wheel tracks is more than the passing lanes due to heavier traffic volume (dotted line in Fig. 36). Fig. 36 illustrates the trend of increase in air permeability with increase in air void content. A low correlation cofficient (r) of 0.58 was obtained. The reason for the poor correlation is related to the differential compaction of the matrix and

closing up of the surface texture by traffic. That is, while the matrix closes up to a lesser degree and allows water to flow through, the tight surface would not. Hence, while the matrix may be open and has high voids, the mixture as a whole could be impermeable as in the case of the wheel tracks in the driving lane.

The influence of percent coarse aggregate (C.A.) on permeability and % voids in pavement was examined. It seems that the increase in % C.A. (e.g. sections 11, 6, and 14) in a mix had increased the permeability and voids in a pavement as shown by the broken line in Fig. 36.

(c) *Asphalt Cement Aging*

This experiment provided a good opportunity to examine the aging properties of the asphalt cement (A.C.) under the same service and environmental conditions for the variety of mixes placed in the test sections. The properties (Table 16) evaluated are as follows:

(i) Penetration ratio and voids in pavement:-

One important piece of information derived from this experiment was the influence of air voids in the pavement on the rate of asphalt cement (A.C.) hardening after 11 years of aging under the same service environment. With the assumption that the original penetration of an 85/100 penetration grade A.C. used was 92 (except section 17 which was assumed at 65 pen.), the retained penetration was obtained by dividing the recovered penetration at the eleventh year by the original A.C. pen.. Fig. 37 confirms the general belief that increase in air voids in the pavements will lead to more rapid oxidation and hardening of the A.C. in a mix and that the relationship is nonlinear.

It seems that a pavement with a void content of, say, 10% will have 10% less retained penetration than a mix with 5% at the 11th year.

(ii) Recovered A.C. properties and transverse cracking:-

The pen. vis. number (PVN), penetration index (PI) and softening point (SP) have been used by some researchers as a means to correlate the temperature susceptibility and the cold temperature cracking potential of asphalt cement and mixtures. These aspects were examined, recognizing the fact that these results were obtained from recovered A.C. rather than from the original materials (for which the PI would likely be in the range of -1.0 to -0.5 for the 85/100 grade AC).

Fig. 38a shows the general trend of increase in cracking with reduction in the PVN (based on McLeod's formula, ref. 8 and 9). It seems that significant transverse cracking could occur when the PVN falls below the level of -0.3. For the open mixes, having a higher range of PVN (ranging from -0.4 to +0.3 versus -0.6 to -0.1 for dense mixes) appear to be more resistant to cracking than dense mixes. Also, the fact that open mixes can better absorb micro-cracks and reheal in hot weather may have help to reduce the degree of cracking.

When dealing with virgin asphalt cements, a high PI means a low cracking potential. However, contrary to this belief, Fig. 38b shows that the recovered A.Cs. with high PI had a high transverse cracking rating. This applied to both dense and open mixes. The PI for dense mixes ranged from +0.4 to +1.9 and for open mixes from +0.9 to +3.8. The curve for the denser mixes (r =0.51) had a greater change than the open mixes (r = 0.76, e.g. sections 3, 6, 7, 9, 10, 13, 14). These mixes had pavement void content of greater than 7%.

Similarly, when relating transverse cracking to softening point (SP, R&B), the trend of more cracking with high SP was obtained (Fig. 38c). When the more open mixes only (e.g. sections 3, 6, 7, 9, 13 and 14) were included in the analysis, the opposite trend seemed to occur. However, the trend (r = 0.62) of greater cracking with high SP appears to be easier to understand in that high SP A.C. tends to be stiffer and generate more cracks. Also, lower SP A.C. would be more likely to have minor cracks healed in the hot weather, leading to lower crack counts.

In general, based on the above analysis, the PVN and SP produced better correlation with the transverse cracking rating than the PI for the A.C.recovered from aged asphalt mixtures. The results also suggest that the PVN is relatively insenitive to changes in an A.C. during the aging process.

(iii) Softening point (SP) and PVN and PI:

A good correlation (Fig. 39a) was obtained when relating the PVN with SP. However, this contradicts the belief that high PVN leads to lower cracking as discussed above. This kind of analysis may, on the other hand, not apply to aged A.C. of11 years old. Similar exercise in relating the PI with SP did not yield any relationship (Fig. 39b). Only a slight trend of increase in PI with increase in SP seemed to exist.

(d) *Recompacted Marshall Properties*

Cores taken from the pavement were reheated and recompacted into Marshall briquettes for testing. The results (Table 17 and Fig. 40a) showed that the stability of samples from the driving lane was generally lower than the centre or the passing lane. This is due to the increased hardening of A.C. in the centre and passing lanes where higher voids were maintained at a longer period of time.The following describes some of the relationships that are related to the Marshall properties.

(i) Change of Marshall stiffness with time:

The Marshall stiffness is obtained by dividing the stability with flow (kN/mm, Fig. 40b). The relationship shown in Fig. 41 confirms the fact that more open mixes (sections 3, 6, 7, 9, 10, 13, 14) had more A.C. hardening leading to higher Marshall stiffness with time. Except section 2, all the mixes had increased in stiffness with time of service. The decrease in stiffness in section 2 could be an indication of the rutting experienced.

(ii) Marshall stiffness and cracking:

The influence of the Marshall stiffness on the cracking potential of asphalt mixes was evaluated. It seems that higher Marshall stiffness mixtures tended to have less transverse tracking (Fig. 42). It seems also that when the stiffness fell below about 3 kN/mm, high transverse cracking could be expected. All the mixes which had low stiffness contained asbestos.

(e) *Tensile Strength and Modulus Characteristics*

(i) Resilient Modulus (M_R):

The M_R results were obtained (Table 17) by testing the core samples at 23^{o}C (Fig. 43a). The mixes that had the highest M_R were sections 5, 3, 12 and 9 and the lowest ones were sections 17, 10 and 6. As mix stiffnesses could be used to evaluate the cracking potential of asphalt mixtures, the analysis (Fig. 44a) of these mixes together with others did not suggest that it is possible in this case.

The relationship of M_R with the coarse aggregate content (% C.A.) was examined (Fig. 45a). By plotting the M_R values for mixes using the same types of aggregate but at various proportions, the graph shows that there was an optimum % C.A. at which the highest M_R

occurred. Different types of aggregate combinations had different peaks ranging between 45 to 55% C.A. content. The mixes with all traprock aggregate seemed to have the highest maximum M_R (e.g. = 4400 MPa) at about 50% C.A..

(ii) Indirect Tensile Strength (S_t):

Similar analysis was carried out for the S_t values (Table 17). Again, there was no relationship between the S_t and the transverse cracking rating. Also, there seemed to be a maximum S_t for each of the groups of mixes using the same aggregate type combination of various C.A. contents (Fig. 45b). The peaks ranged from 35 to 45% C.A. content.

The MR and S_t relationships with the %C.A. confirm that the conventional type of dense mixes with about 45% of retained 4.75 mm sieve provide better strength properties for structural purposes than the open mixes.

(f) *Gyratory Testing Machine (GTM) Properties*

The GTM test was performed on the cores taken from the pavement. For the mixes with an insufficient amount of material from one lane, materials from different lanes were combined for the test. Tests were performed at 345 kPa (50 psi), 552 kPa (80 psi) and 827 kPa (120 psi) vertical pressures and at different revolutions. The two most relevant parameters, i.e. gyratory shear strength (Sg) and gyratory stability factor were examined in the manner as suggested in the Report EM-78 (10). However, the criteria as set forth in the report were not used for determining the stability of the mixes for rutting because the cores were aged samples from the road and had been reheated for the test. Despite that, the results were used to study the trend and behaviour of the different mixes.

(i) Gyratory Shear Strength (Sg):

All of the sections (except section 14) had an increase in Sg with increase in BRD (Fig. 46). This indicates that closer aggregate interlock and better strength were obtained. The decrease in Sg with increase in BRD for section 14 could be due to the degradation of the aggregates as a result of over compaction of the mix to a voidless state and a flushed condition.

The results of sections 3, 13, and 15 confirm the general belief that an increase in stone content will improve the shear strength

property of a mix (Fig. 46). Also, the use of natural sand in a mix tends to reduce the Sg values (i.e. by comparing sections 10 with 9).

(ii) Gyratory Stability Index (G.S.I.):

It appeared that all of the stable mixes had maintained their G.S.I. with the increase in BRD except sections 1, 2, 5,11, 14, 15, 16 and 17. There was one common trend existed between all these mixes i.e. at the maximum relative density, the mixes had dramatic increase in their GSI values (Fig. 42). The following explains some of the defficiencies of each of the potentially poor performer mixes:

- section 1,2 and 5 had natural sand and asbestos.
- section 11 & 16 had high asphalt cement content and asbestos.
- section 17 had mineral filler, asbestos and high asphalt cement content.

7.4 SUMMARY OF FINDINGS

(a) *Frictional Performance*

i) The analysis of the long term frictional performance of the mixes confirms the initial findings reported previously by Ryell et al (2) that: - for high speed road overlays, good friction develops from surfaces with sufficient macrotexture and harsh microtexture.
- the dense mixes using slag or traprock fine aggregates provided better friction properties than limestone screenings or natural sand.

ii) All mixes decrease in the Friction Number (FN) within the first two to four years before arriving at their equilibrium values. Mixes with high initial FN tend to take longer to reach the equilibrium.

iii) Although sections 11 and 12 produced a reasonably high level of friction at speed of 50 km/h, the rapid drop in FN at higher speed (i.e.100 km/h) shows that there was inadequate macrotexture. On the other hand, the benefit of the more open macrotexture mixes (sections 13 and 14) was realized with the reduction in the drop of the FN and speed curve gradient.

iv) There is a good correlation between the commonly known sand patch texture depth and the photointerpretation texture height

parameter 'A' by ASTM D770 method. A good sand patch texture depth of 0.6 mm was obtained after 11 years of traffic in the driving lane from an OFC mix containing 67% of coarse aggregate.

v) For the best frictional properties, such as steady FN to speed gradient and good texture depth and density, the results confirm that coarse aggregate content should be set above 60%.

(b) *Durability Performance*

i) All of the 18 test sections performed better than expected for single course thin overlays on concrete pavement.

ii) In the case of thin overlays on concrete pavement, results did not suggest any difference in crack generation and joint repair requirement between dense and open mixes; but there is a slight trend that high stone content open mixes tend to have more ravelling along the cracks.

iii) The use of asbestos may contribute to more cracking as it reduces the binder film thickness and flexibility. Increased binder film thickness and coarse aggregate content had reduced the cracking potential of asphalt mixtures tested.

iv) Poor bonding between the concrete pavement and the surface course mixes, as a result of inadequate tack coating and/or binder film, had lead to extensive delamination on the driving lane of sections 9 and 14. High stone content mixes tend to have increased delamination. On overlaying concrete pavements in particular, good bonding created by either proper tack coating or the use of a denser binder course was found essential for promoting good durability.

v) The blast furnace slag mix experienced more ravelling than other non-absorptive aggregate mixes.

vi) Standard mix with marginal stability rutted.

vii) It was found that coarse aggregate (C.A.) content had a definite effect on the overall durability performance of the asphalt mixes, particularly relating to parameters such as ravelling, delamination and transverse cracking. Higher C.A. content mixes tend to have lower cracking but more ravelling and delamination.

(c) *Material Properties*

i) After 11 years of heavy trafficking, the all-slag mixes had the highest void content (average 10%) among the test mixes.

ii) The OFC mixes had very high initial permeability which was reduced to a level similar to that for dense mixes after seven years of traffic compaction. The permeability of the DFC mix was reduced to a level of a dense mix after only two years of heavy trafficking. As expected, an increase in the % C.A. had generally increased the permeability and voids of asphalt mixes.

iii) The results of this study confirmed that increase in air voids in the pavements had lead to more rapid oxidation and hardening of asphalt cement in asphalt mixes. A 10% decrease in the retained of the original penetration (i.e. more hardening) can be expected with the increase in void content of a pavement from 5 to 10% as measured at the eleventh year of service.

iv) Based on this study of using recovered A.C. from aged asphalt materials, the PVN and softening point (SP) produced better correlation with transverse cracking rating than the penetration index (PI).

v) Mixes with a low Marshall stiffness of below 3 kN/mm had high transverse cracking. In this study, the low stiffness was associated with mixes containing asbestos.

vi) For good structural properties as shown by M_R and S_t, the % C.A. of dense asphalt mixes was found to be around 45%. However, from the point of view of durability, the % C.A. should be set above or below the range of 35 to 45%.

vii) The gyratory testing machine results confirmed that increased stone content will improve the shear strength of asphalt mixtures.

8. CONCLUSIONS

All of the test sections performed better than expected for single course thin overlays on concrete pavement. The open and dense mixes performed equally well under the heavy traffic prevailed.

In addressing the objective of this experiment i.e. the search for

bituminous mixes with good frictional performance, the most suitable mixtures for overlaying high speed and heavy traffic highways appears to be those which contain:

a) steel slag or traprock aggregates for both the coarse and fine aggregates.
b) coarse aggregate contents of above 60 %.
c) sufficient binder film thickness and voids.

From the mix design point of view and within the context of this trial, the findings which are considered significant are:

i) coarse aggregate contents above 60% provided the optimum friction properties.
ii) 45% coarse aggregate content produced the highest resilient modulus and indirect tensile strength.
iii) for durability, the % C.A. should be above or below the range of 35 to 45% and the existing pavement surface should be properly tackcoated, especially concrete pavements.
iv) when Marshall stiffness falls below 3 kN/mm, high transverse cracking can be expected.
v) open mixes have lower cracking ratings than dense mixes.
vi) a 10% increase in hardening can be expected with the increase in pavement void content from 5 to 10% over an eleven year period.
vii) proper gradation design and aggregate selection is the key to solving the rutting problem.

The findings of this experiment confirmed that the existing polices of using OFC and DFC for treatment of highways requiring friction surface course are sound and adequate.

9. RECOMMENDATIONS

(1) The Ministry should continue the policies of using OFC and DFC mixes for heavily travelled pavements.

(2) For OFC mixes, a coarse aggregate content of 65% minimum should be maintained.

(3) For DFC mixes, a coarse aggregate content of 55% minimum should be maintained.

REFERENCE

(I) Corkill, J.T., "Construction of I7 Test Sections of Special Bituminous Mixes". Paper presented at the Annual Conference of the Canadian Technical Asphalt Association, Nov., I975.

(2) Ryell, J., Corkill, J.T. and Musgrove, G.R., "Skid Resistance of Bituminous Pavement Test Sections Highway 40I, Toronto By-Pass. Paper presented at the 673rd Annual Conference of Road and Transportation Association of Canada", Oct., I977.

(3) Schmidt, R.J., "A Practical Method for Measuring the Resilient Modulus of Asphalt-Treated Mixes", Highway Research Record #404, I972.

(4) Emery, J.A. and Lee, M.A., "Layer Coefficients Based on Resilient Modulus Tests". A restricted report prepared for MTC, I977.

(5) ASTM D 4123-82 "Indirect Tension Test for Resilient Modulus of Bituminous Mixtures " ASTM Vol.04.03 Road and Paving Materials; Traveled Surface Characteristics.

(6) Holt, F.B. and Musgrove, G.R., "Surface Texture Classification: A Guide to Pavement Skid Resistance", ASTM STP 763, I982.

(7) Chong, G.J., Phang, W.A. and Wrong, G.A., "Manual for Condition Rating of Flexible Pavements", Ministry of Transportation and Communications, SP004, I982.

(8) Plummer M.A. and C.C. Zimmerman, "Asphalt Quality and Yield Predictions from Crude Oil Analysis", Proceedings of the Association of Asphalt Paving Technologists, Vol. 53, I984.

(9) Puzinaskas V.P., "Properties of Asphalt Cements", Proceedings of the Association of Asphalt Paving Technologist, Vol. 48, I979.

(10) Tam, K.K. and Lynch, D.F., " Evaluation of the Rutting Potential of Asphalt Mixes Using the Gyratory Testing Machine", Ontario Ministry of Transportation and Communications, Engineering Materials Report EM-78, 1986.

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Table 1: TRAFFIC VOLUME AND TRUCK WEIGHT

YEAR	AADT	GVW (tonnes)	LEGAL AXLE WEIGHT (tonnes)
1963	77,000	32.0	8.6
1974	179,550	63.5	10.6
1985	260,600	63.5	10.6

TABLE 2: WEST BOUND CORE LANE TRAFFIC SPLIT DATA

YEAR	DRIVING LANE AADT	% TRUCK	CENTRE LANE AADT	% TRUCK	PASSING LANE AADT	% TRUCK
1975	12,900 (29)	65	17,300 (39)	33	14,600 (32)	2
1985	19,431 (34)	--	20,382 (35)	--	17,887 (31)	-

Note: ( ) = % of total core lane traffic.

TABLE 3: BITUMINOUS MIXES, COMPOSITION AND DESIGN DATA

Test Section Number	Mix Type	Mix Composition % Wt. of Aggregate Plus Filler: Coarse Agg. Ret. No. 4	Fine Agg. Pass No. 4	Filler Material	% Ret #4 *	Asphalt % Wt of Mix	Mix Characteristics: Marshall Stability	Marshall Flow	V.M.A. % Vol.	Voids % Vol.	
1	HL1	45TR	41TRS		43.8	5.4	1625	10.3	16.9	2.8	L
			14LS			5.4	2614	11.6	17.5	3.7	F
2	HL1	45TR	41NS		48.2	5.4	1625	10.3	17.2	3.0	L
			14TRS			5.3	2870	13.4	15.0	0.8	F
3	HL1	45TR	55TRS		47.5	4.1	3185	16.5	12.9	1.2	L
						4.0	3407	15.3	13.5	2.0	F
4	HL1	55TR	34NS		54.1	4.8	1730	10.2	16.0	2.7	L
			11LS			4.8	3027	12.3	14.2	1.2	F
5	HL1	60TR	28NS	2ASB	58.1	5.6	1800	15.0	14.2	1.0	L
			10LS			5.7	1903	22.5	16.0	0.9	F
6	HL1	60TR	38TRS	2ASB	62.3	5.3	2190	22.7	15.2	1.0	L
						5.4	2570	18.7	18.6	3.4	F
7	Mod. HL1	45SL	55SLS		46.8	5.3	3520	16.0	18.5	2.7	L
						5.2	3640	13.2	18.7	3.3	F
8	Mod. HL1	50SL	38NS		47.1	5.7	2160	12.9	17.3	2.3	L
			12LS			5.7	2990	13.7	17.7	1.9	F
9	Mod. HL1	45BF	55BFS		43.2	8.0	2830	12.8	24.0	6.9	L
						7.8	3328	14.9	23.1	6.1	F
10	Mod. HL1	40BF	45NS		40.5	6.0	1975	10.4	17.9	2.9	L
			15LS			6.5	3155	9.7	17.0	2.2	F
11	Sand Mix	14TR	84TRS	2ASB	5.4	7.1	2084	27.2	17.7	0	L
						7.0	2175	40.2	19.5	0.2	F
12	Sand Mix	9TR	89TRS	2ASB	6.9	7.0	2650	19.2	16.8	0	L
						7.2	1885	45.1	20.7	1.1	F
13	Open Graded	87TR	33TRS		80.5						
						5.9	1458	12.7	20.8	4.7	F
14	Open Graded	67TR	31TRS	2ASB	71.7						
						5.8	1691	11.9	19.6	4.0	F
15	Open Graded	30TR	70TRS		29.3						
						5.6	2678	19.2	16.3	0.7	F
16	Open Graded	30TR	68TRS	2ASB	31.4						
						6.6	2116	30.5	18.6	0.2	F
17	Mastic	70TR	19TRS	9MF 2ASB	76.2						
						7.5	1887	56.7	21.0	1.0	F
**18	HL1	45TR	41NS		47.4	5.4	1625	10.3	16.9	2.8	L
			14LS			5.4	2825	14.5	14.6	0.4	F

* Based on field laboratory extraction tests.

** As Test Section 1 but constructed over a 38 mm (1½") thick bituminous base course layer.

LEGEND

Coarse Aggregate
TR - Traprock
SL - Steel Slag
BF - Blast furnace Slag

Fine Aggregate
SLS - Steel Slag Screenings
LS - Limestone Screenings
TRS - Traprock Screenings
BFS - Blast Furnace Screenings
NS - Natural Sand (Glacial Deposit)

Filler Material
ASB - Short Fiber Asbestos
MF - Mineral, Filler (Finely Crushed Limestone)
L - Laboratory Mix Designs
F - Field Laboratory Tests

Table 4: Type of Field and Laboratory Tests Performed for Different Monitoring Years

Year	0	1	2	3	4 (a)	5	6 (b)	7	10	11 (c)
Field Test										
* ASTM Trailer: Spring		x	x	x	x		x	x		
Fall	x	x	x	x	x	x			x	x
*Photointerpretation	x	x	x					x	x	x
*Water Permeability	x	x	x		x				x	
*Rutting								x	x	x
*Pendulum								x		
Laboratory Test										
*Marshall Recompactn.		x						x		x
*Gradation	x							x	x	x
*Pav't Compaction	x	x						x	x	x
*Pavement Voids	x	x						x		x
*Air Permeability									x	x
*A.C. Penetration									x	x
*A.C. Viscosity									x	x
*A.C. Soft. Point									x	x
*Resil. Modulus (MR)									x	x
*Ind. Tensile Str.										x
*G.T.M.										x
*Sand Patch										x

Note: (a) : Permeability test on cores from section 13 only.
(b) : Cores from sections 1 to 4 only.
(c) : Performed pavement condition survey.
x : Test Performed

Table 5: Pavement Condition Rating of Test Sections after 11 Years of Service

Section	Type of Distress						
	Cracks						
	Trans.	Long.	Ravel.	Patch	Del	Rut.	Overall Rating
1	3.5	44.0	6.0	0	0	3.0	56.5
2	3.5	3.0	12.0	0	0	18.0	36.5
3	0	1.0	18.0	6.0	4.0	3.0	32.0
4	11.5	3.7	6.0	0	0.3	3.0	24.5
5	44.5	7.0	18.0	4.5	2.8	0	76.8
6	3.0	0	12.0	7.5	5.5	0	28.0
7	16.0	2.0	6.0	0.8	1.0	3.0	28.8
8	21.5	7.8	0	0	0.2	4.5	34.0
9	5.0	7.0	18.0	5.5	14.0	4.5	54.0
10	10.5	6.5	6.0	0	3.5	4.5	31.0
11	33.5	0	0	0	2.0	0	35.5
12	48.0	5.8	6.0	0	1.2	0	61.0
13	6.0	2.0	6.0	0	4.5	0	18.5
14	4.0	1.0	18.0	15.0	3.0	0	41.0
15	19.5	2.0	6.0	0	0.2	0	27.7
16	39.5	17.5	6.0	0	4.0	0	67.0
17	25.5	5.0	18.0	1.5	1.5	0	51.5
18	3.8	4.0	0	0	1.0	6.0	14.8

Table 6: Length of Joint Repairs (m)

Section	Trans. Cr.	Long. Cr.	Total	shoulder	
				Left	Right
1	41.4	7.5	48.9	0	0
2	52.7	1.9	54.6	57.9	0
3	86.6	3.8	90.4	115.8	0
4	47.1	0	47.1	15.2	0
5	49.0	26.4	75.4	121.9	0
6	50.8	60.3	111.1	128.0	0
7	39.5	5.6	45.1	137.2	0
8	-	-	-	-	-
9	26.4	15.1	41.5	0	0
10	54.6	0	54.6	0	0
11	56.5	3.8	60.4	0	0
12	16.9	3.8	20.7	0	27.4
13	18.8	3.8	22.6	137.2	7.6
14	0	7.5	7.5	137.2	0
15	0	0	0	137.2	45.7
16	16.9	0	16.9	137.2	0
17	3.8	0	3.8	73.2	0
18	0	3.8	3.8	0	0

Table 7. Rut Depth Measurement (mm) of Test Sections on the Wheel Tracks at 11 Years

Test Sect	Driving Lane			Center Lane			Passing Lane		
	RWT	LWT	Avg.	RWT	LWT	Avg.	RWT	LWT	Avg.
1	0	0	0	0	0	0	0	5	3
2	13	17	15	3	3	3	3	4	3
3	3	3	3	2	2	2	0	4	2
4	2	0	1	0	0	0	0	0	0
5	0	0	0	0	0	0	4	4	4
6	0	0	0	2	0	1	4	4	4
7	0	0	0	4	0	2	2	2	2
8	3	3	3	5	5	5	3	3	3
9	4	4	4	4	4	4	0	0	0
10	4	4	4	5	4	5	4	4	4
11	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0
13	0	0	0	0	0	0	0	0	0
14	0	0	0	0	0	0	0	0	0
15	2	2	2	0	0	0	0	0	0
16	0	0	0	0	0	0	0	0	0
17	0	0	0	0	0	0	0	0	0
18	5	7	6	4	6	5	0	0	0

Note : RWT = right wheel track
LWT = left wheel track
Each value is the average of 2 measurements.

Table 8 : Development of Friction Number (FN50 and FN100) of Different Test Sections with Age of Pavement

SECTION		1		2		3		4		5		6		7		8		9		10		11		12		13		14		15		16		17		18	
DATE	LOC	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100	50	100
Aug_74	DRIVING LANE	51	32	50	34	60	45	54	37	48	31	48	37	62	48	61	38	50	37	48	37	50	38	45	29	43	42	41	37	51	40	47	37	41	28	43	33
Oct_74		42	28	41	25	51	39	43	33	44	31	47	36	53	42	42	30	54	42	46	25	46	31	44	23	50	41	48	37	53	39	50	32	41	27	38	32
Apr_75		43	26	46	28	45	37	43	30	41	29	47	35	50	40	47	30	51	39	44	30	49	30	46	30	47	38	46	38	49	35	47	34	42	31	43	30
Oct_75		33	22	36	25	41	35	34	27	35	26	39	35	46	39	37	25	45	38	35	26	41	29	39	27	40	37	39	36	39	28	40	30	35	30	34	23
May_76		33	25	40	27	42	36	36	25	36	25	44	35	44	37	40	28	44	37	34	25	42	28	42	31	40	36	40	35	42	29	42	32	36	30	32	22
Nov_76		33	22	36	24	39	32	35	28	36	25	39	31	42	36	37	26	41	34	34	25	40	29	40	30	39	32	42	30	40	30	35	28	32	22
July_77		30	22	32	22	34	27	31	24	33	24	35	27	46	34	36	26	43	32	33	24	36	28	36	28	34	31	35	29	35	25	35	27	33	27	30	21
Nov_77		30	23	31	23	33	29	33	27	32	25	33	28	41	35	36	27	41	33	33	25	37	27	37	28	32	30	32	30	35	26	36	27	32	26	30	23
July_78		29	25	30	25	33	29	31	26	31	26	34	30	41	35	34	28	41	34	33	26	37	29	37	29	34	31	34	31	35	28	35	29	31	27	29	23
Nov_78		29	21	31	21	34	27	31	25	33	24	34	29	41	33	34	27	40	33	33	25	36	28	35	27	33	29	33	30	34	26	35	27	31	27	29	20
July_79		34	23	37	24	40	29	36	27	36	27	40	30	49	36	43	28	46	32	40	26	44	28	44	29	39	31	38	30	40	26	42	29	36	27	35	21
Oct_79		32	25	33	24	34	28	33	26	33	25	35	29	44	35	38	28	43	35	37	27	39	30	38	31	33	30	33	29	35	27	37	29	33	26	31	22
Aug_82		33	20	33	18	37	26	33	23	33	23	37	27	47	29	42	23	47	28	38	21	43	24	43	24	35	26	35	27	37	22	35	23	34	23	30	15
Aug_84		34	25	35	24	39	25	34	25	35	30	38	27	47	35	41	22	48	37	39	29	44	30	43	31	37	30	36	31	38	25	36	29	35	21	31	29
Aug_85		35	25	36	23	40	31	35	28	37	28	39	32	47	36	40	22	49	36	41	29	45	32	44	30	39	32	37	33	40	28	41	29	37	21	33	21
Aug_74	CENTRE LANE	49	36	50	35	62	47	55	39	47	31	48	38	61	46	57	33	52	38	48	38	50	41	49	38	45	43	44	40	44	43	50	41	40	30	41	33
Oct_74		41	33	46	34	53	44	46	35	40	30	47	35	54	43	47	33	52	46	45	35	50	39	50	32	48	42	44	36	51	43	50	36	39	29	39	27
Apr_75		38	30	45	30	47	39	43	30	40	29	45	36	51	42	45	30	49	47	45	34	45	36	43	30	47	40	44	37	49	44	47	38	37	29	34	25
Oct_75		33	27	44	26	44	35	41	27	38	28	47	35	50	39	40	28	46	41	39	30	41	33	46	30	47	38	44	37	44	34	45	33	36	28	33	25
May_76		33	27	42	30	42	33	36	30	38	29	46	36	48	39	40	28	45	39	38	30	43	31	45	30	45	34	41	35	42	34	43	33	35	29	33	25
Nov_76		33	26	41	28	41	33	39	29	37	27	41	33	44	37	40	28	45	35	36	28	41	31	45	31	44	34	40	32	43	31	43	33	37	29	33	25
July_77		27	23	33	24	36	29	34	26	33	28	36	29	42	33	32	23	37	34	35	29	37	29	37	28	35	29	33	30	35	28	36	27	32	26	30	26
Nov_77		34	24	36	23	37	28	34	26	34	26	35	29	45	34	37	28	41	34	37	26	40	29	40	27	34	29	35	30	37	28	37	28	34	26	33	23
July_78		33	26	35	27	38	30	35	27	35	27	36	29	44	35	35	29	42	34	36	28	40	31	40	32	36	31	37	31	38	31	34	28	33	24
Nov_78		31	22	33	24	34	28	32	25	32	24	33	28	40	31	33	27	40	34	33	27	37	29	38	28	33	29	33	26	34	27	36	28	32	26	30	21
July_79		38	22	39	24	40	28	38	26	38	25	40	28	48	33	43	27	46	35	40	28	46	30	46	29	40	30	40	29	43	28	44	29	37	26	35	22
Oct_79		32	23	34	25	36	29	33	27	34	27	35	29	45	34	36	28	42	37	39	29	41	31	42	30	35	31	34	30	37	29	39	30	32	27	31	23
Aug_82		37	20	36	21	38	26	34	24	33	23	39	28	47	29	42	27	44	30	39	23	44	27	44	27	38	27	37	28	40	24	40	26	36	24	33	19
Aug_84		37	24	37	26	39	31	36	28	36	29	39	31	47	35	42	26	47	38	43	29	46	32	46	32	39	32	38	31	40	29	42	31	38	23	35	28
Aug_85		36	26	38	27	41	34	38	31	38	30	40	34	48	38	41	26	51	44	46	32	48	35	47	34	40	33	39	34	41	30	43	32	40	30	36	25
Aug_74	PASSING LANE	49	36	51	36	63	47	57	40	50	40	50	37	63	49	60	36	57	40	50	37	54	40	51	30	51	41	45	36	58	44	54	41	37	28	46	30
Oct_74		45	31	48	30	55	43	46	34	41	32	47	35	53	44	44	26	54	47	47	39	57	40	47	30	46	43	46	39	51	45	51	42	38	22	42	26
Apr_75		43	31	46	34	51	43	43	34	41	33	47	37	52	44	45	32	51	47	45	38	46	39	47	30	49	43	44	40	50	41	49	43	37	22	34	26
Oct_75		37	30	46	32	50	42	40	34	41	32	45	37	50	44	44	30	48	45	45	36	43	36	48	33	49	42	44	39	50	39	49	40	37	26	34	29
May_76		33	27	42	32	51	40	37	32	39	30	40	37	50	42	45	30	46	40	45	34	43	32	46	33	49	35	44	37	47	36	49	38	36	26	35	26
Nov_76		34	26	38	29	45	36	40	30	39	29	40	32	46	40	40	29	46	37	42	33	41	32	46	29	46	34	40	34	46	34	46	35	36	25	35	26
July_77		29	24	36	29	41	36	39	32	40	32	39	33	46	40	37	27	48	43	42	33	37	34	39	30	46	35	38	34	40	33	42	34	35	27	38	32
Nov_77		35	27	37	29	40	33	37	29	37	27	38	30	50	35	38	27	43	35	40	31	44	32	44	31	40	31	37	31	44	31	44	33	37	25	36	26
July_78		35	28	38	31	44	37	39	32	38	32	40	35	47	40	35	27	45	40	37	34	44	35	44	34	40	37	40	36	44	36	46	36	35	27	35	27
Nov_78		34	26	35	28	40	33	36	30	35	29	37	31	47	35	35	23	42	35	38	31	41	31	41	30	37	32	36	31	40	31	41	29	34	25	33	25
July_79		41	26	42	28	47	33	45	29	43	29	45	31	52	36	42	25	49	37	44	31	50	32	50	31	44	33	45	32	49	32	50	31	40	25	39	25
Oct_79		37	27	38	29	42	33	38	29	39	30	39	32	49	38	38	26	46	38	42	32	43	33	45	32	39	34	38	33	43	33	44	34	37	26	37	26
Aug_82		39	24	39	26	43	31	39	28	40	28	43	30	53	35	40	25	49	35	44	28	47	30	48	28	43	33	41	32	45	30	47	30	38	24	36	24
Aug_84		38	27	39	30	45	34	41	31	41	29	43	33	52	38	40	26	49	40	45	32	48	32	43	32	43	34	41	34	45	31	48	32	39	27	37	26
Aug_85		37	29	40	32	47	38	43	34	42	34	43	38	51	41	39	29	50	44	47	35	50	37	49	35	43	37	42	35	45	33	49	35	40	30	38	28

Table 9: Equilibrium Friction Number (EFN) at 50 and 100 km/h of the 18 Test Sections

Test Section No.	DRIVING LANE		CENTRE LANE		PASSING LANE	
	FN50	FN100	FN50	FN100	FN50	FN100
1	32	23	34	23	36	26
2	33	23	36	25	38	29
3	36	28	38	29	43	34
4	33	26	35	27	40	30
5	34	26	35	27	39	30
6	36	29	37	29	41	33
7	45	34	45	34	50	38
8	38	26	38	27	38	25
9	44	33	44	36	47	39
10	36	25	39	28	42	32
11	40	28	42	30	45	33
12	40	29	42	30	45	31
13	35	30	37	30	42	34
14	35	30	36	30	40	33
15	37	26	38	28	44	32
16	37	28	39	29	46	33
17	34	25	35	26	37	26
18	31	22	33	23	37	27

TABLE 10

TEXTURE DEPTH BY THE SAND PATCH METHOD TESTED ON 6" DIA.CORES (at 11 yrs)

Test Section No.	TEXTURE DEPTH (mm)								
	Driving Lane			Center Lane			Passing Lane		
	LWT	ML	RWT	LWT	ML	RWT	LWT	ML	RWT
1	.54	.28	.43	.33	.32	.21	.62	.31	.38
2	.29	.27	.3	.56	.38	.27	.57	.38	.47
3	.59	.4	.74	.9	.44	.67	.8	.48	.61
4	.3	.45	.52	.58	.44	.53	.63	.43	.42
5	.53	.39	.55	.76	.34	.5	.89	1.05	.52
6	.65	.4	.83	1.07	.51	.68	1.39	1.17	.67
7	.37	.32	.3	.54	.31	.53	.62	.67	.57
8	.33	.32	.31	.42	.28	.3	un	un	un
9	.45	.31	.46	.71	.4	1.1	.89	.44	.6
10	.36	.22	.38	.33	.25	.48	.54	.35	.46
11	.25	.22	.31	.32	.32	.34	.6	.26	.27
12	.26	.22	.25	.34	.3	.33	.37	.16	.22
13	.6	un	.9	.69	.61	.84	1.04	.64	.85
14	.77	.64	.93	1.01	.55	1.03	1.57	.79	.96
15	.2	.12	.39	.4	.25	.35	.47	.25	.35
16	.33	.29	.41	.47	.31	.36	.47	.36	.33
17	.62	.38	.66	.64	.53	.56	.76	.64	.54
18	.3	.26	.25	.28	.31	.29	.53	.33	.34

NOTES:

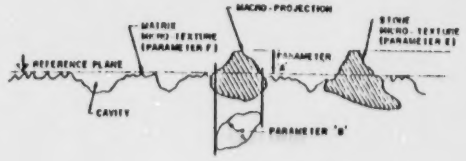
1. LWT= Left Wheel-Track, ML= Center of the lane
   RWT= Right Wheel-Track, un= result not available
2. Results related to wheel-tracks are averages of 2 cores.

(a) Macrotexture Parameters

Test Section No.	DRIVING LANE										CENTRE LANE								PASSING LANE									
	August 74			May 76			May 85				November 84				May 85				August 74			May 76			May 85			
	A	B	D	A	B	D	A	B	C	D	A	B	C	D	A	B	C	D	A	B	D	A	B	D	A	B	C	D
1	.43	3	20	0	0	0	.35	3.92	2.2	42.5	.58	3.82	1.9	42.2	1.02	4.41	1.7	55.7	.38	4	32	.43	3.8	23	1.26	4.29	1.8	41
2	.30	4.8	30	0	0	0	.32	3.82	1.9	38.7	.41	3.49	1.7	35	.7	4.02	1.5	46.3	.45	4	28	.65	4	38	1.24	4.86	1.8	49.3
3	1	4.4	72	.8	3.8	37	.70	3.65	2.2	48.5	1.21	4.1	2.2	52.2	1.64	4.22	2	68.8	1	4.4	78	.8	4	75	2.07	4.5	2.5	80.7
4	.65	4.8	32	.7	3.8	22	.86	4.2	2.1	49	.79	4.29	2	48	1.53	5.19	1.9	52	.75	4.4	50	.5	3.8	30	1	3.92	2.3	57.5
5	.65	6	25	.4	4	15	1.05	4.31	2.1	56.2	.69	4.49	2.1	57.2	1.7	4.6	2.2	69.5	.6	6.8	35	.6	7.2	27	1.42	4.5	2.3	66.7
6	1	4.8	42	.75	4	32	1.13	3.91	2.2	55.5	.88	4.39	2.2	51.3	2.52	4.39	2.2	75.8	.95	4.8	50	.7	5.6	57	2.16	5.41	2.4	82.3
7	.95	3.8	52	.6	4	24	.27	3.18	1.6	33	.3	3.56	1.3	30	1.05	3.82	1.7	47.3	.95	3.4	58	.75	3.2	45	1.48	3.82	2.2	69.5
8	.8	4	27	.45	3.6	15	.3	3.43	1.5	31.1	.29	2.92	1.3	24.7	.36	3.46	1.7	34.3	.55	4	30	.45	4	23	1.12	3.49	2.1	49.1
9	.6	4	42	.43	4.8	25	.63	4.41	1.4	43.7	.32	4.6	1.2	52.6	.97	4.41	1.9	56	.7	3.8	45	.48	3.2	45	1.84	5.02	2.3	72.2
10	.65	4.8	27	0	0	0	.48	5.3	1.3	49	.31	3.2	1.3	45.8	.52	2.91	1.6	36.5	.65	4.8	30	.48	4	30	.65	3.42	1.6	43.7
11	.6	2.8	24	0	0	0	.56	2.77	1.5	30.1	.42	2.71	1.5	24.1	.53	2.75	1.7	31.6	.45	3.6	35	0	0	0	.68	2.47	2.5	34.7
12	0	0	0	0	0	0	.38	2.55	1.4	23.5	.36	2.52	1.4	25	.48	2.41	1.8	29.8	0	0	0	0	0	0	.81	2.41	2.4	35.2
13	NA	NA	NA	.9	6	45	1.79	4.93	2	81.5	1.7	4.83	2	79.3	2.19	4.05	2.5	83.5	1.3	6	45	1	5.6	45	2.16	5.08	2.5	87.3
14	.8	2.8	40	.65	3.8	35	1.74	5.04	1.9	75.8	1.53	4.93	2	73.7	2.05	4.83	2.4	78.7	.75	3.6	45	.75	3.6	45	2.59	5.79	2.5	86.7
15	0	0	0	0	0	0	.55	2.93	1.4	28.1	.37	3.33	1.4	29	.9	3.6	2	42.3	0	0	0	0	0	0	1.27	3.5	2.4	64.8
16	.48	3	25	.45	3.2	15	.67	3.18	1.6	33.5	.41	3.33	1.6	22.5	.87	3.16	1.8	30.7	0	0	0	0	0	0	1.56	4.02	2.6	80.2
17	.9	5.2	70	1	5.2	65	1.33	4.73	2	60	1.36	4.39	2.3	62.3	1.4	4.81	2.3	64.5	.7	6	75	.65	6	72	1.72	5.21	2.4	74.2
18	.6	2.4	18	0	0	0	.49	4.06	1.4	45.7	.26	3.33	1.1	36.2	.28	3.65	1.1	39.2	.6	3.2	12	.33	3.4	15	.94	3.65	2.2	47.8

(b) Microtexture Parameters

Test Section No.	DRIVING L.		CENTRE LANE				PASSING L.	
	May 85		Nov.84		May 85		May 85	
	E	F	E	F	E	F	E	F
1	3	3.8	2.6	3.4	2.6	3.4	2.5	3.7
2	3.2	3.7	2.6	3.4	2.8	3.5	2.5	3.6
3	3.1	3.9	2.3	3.6	3	3.7	3	3.8
4	3	3.7	2.7	3.7	3	3.7	2.9	3.7
5	3	3.8	3	3.7	3	3.5	3	3.8
6	3.1	3.8	3	3.6	2.9	3.5	3.1	3.7
7	3.1	3.9	3.1	3.9	3	3.8	2.9	3.8
8	3	3.8	3.1	3.9	3.1	4	2.8	3.6
9	3.2	4	3.8	4	3.3	4.1	3.3	3.8
10	3.3	3.9	2.9	3.9	3	4	3.5	3.8
11	2.9	3.9	2.8	3.8	3	4	3	3.8
12	2.9	3.9	2.8	3.9	3.1	4.1	2.9	3.8
13	2.8	3.5	2.8	3.5	3	3.5	2.8	3.5
14	2.8	3.6	2.8	3.5	3	3.5	2.8	3.6
15	2.8	3.8	2.8	4	3	4.1	2.8	3.6
16	2.9	3.9	2.8	3.9	3.1	4.1	2.9	3.8
17	2.9	3.4	2.8	3.4	2.9	3.3	3.1	3.3
18	2.8	3.6	2.7	3.6	2.8	3.9	2.9	3.5



'A' - Height of macro projections (mm)
'B' - Width of macro projections (mm)
'C' - Angularity of macro projections (scale 00 - 370)
'D' - Density of distribution of macro projections (%)
'E' - Hardness of macro projection surfaces (scale 00 - 60)
'F' - Hardness of matrix projections on matrix surface (scale 00 - 60)

Table 11: Macrotexture and Microtexture Parameters by Photointerpretation Method (ASTM E770) at Different Age of the Pavement in the Three Lanes

Table 12: British Pendulum Value of Pavement Surfaces Measured after 7 Years of Service in the Driving Lane

Section	Right Wheel Track (R.W.T.)	Between Wheel Tracks
1	54	55
2	56	55
3	51	59
4	52	56
5	49	53
6	51	57
7	58	un
8	57	65
9	57	66
10	57	63
11	58	62
12	57	62
13	47	57
14	46	52
15	53	57
16	53	57
17	45	55
18	47	48

un = Result not available

Table 13: Comparison of Friction Number (FN 100) Values as Obtained by Using Equation by Holt and Musgrove and Actual Measurements

Test Section No.	DRIVING LANE		CENTRE LANE		PASSING LANE	
	Calculated	Measured	Calculated	Measured	Calculated	Measured
1	37.3	25	38.0	26	41.2	29
2	36.3	23	34.0	27	41.9	32
3	42.3	31	44.3	34	54.6	38
4	41.4	28	45.2	31	43.7	34
5	43.4	28	49.9	30	47.0	34
6	45.3	32	57.3	34	53.8	38
7	34.5	36	42.4	38	47.9	41
8	33.5	22	35.8	26	45.4	29
9	37.3	36	46.1	44	54.0	44
10	35.2	29	36.0	32	37.8	35
11	36.0	32	36.0	35	43.1	37
12	33.9	30	36.4	34	44.6	35
13	47.7	32	54.1	33	54.7	37
14	47.1	33	52.4	34	58.9	35
15	34.7	28	41.3	30	47.8	33
16	37.5	29	39.9	32	51.7	35
17	43.9	21	45.5	30	49.2	30
18	33.0	21	29.0	25	42.3	28

Table 14: Pavement Densities and Voids Content Data for the 18 Test Sections

Test Section	DRIVING LANE			CENTRE LANE			PASSING LANE			AVERAGE (ALL LANES)		
	MRD	BRD	VIP	MRD	BRD	VIP	MRD	BRD	VIP	MRD	BRD	VIP
1	2.656	2.597	2.2	2.711	2.646	2.4	2.697	2.569	4.7	2.688	2.604	3.1
2	2.752	2.662	3.3	2.740	2.676	2.3	2.762	2.687	2.7	2.751	2.675	2.8
3	2.953	2.725	7.7	2.974	2.651	10.9	3.004	2.665	11.3	2.977	2.680	10.0
4	2.792	2.702	3.2	2.774	2.689	3.1	2.763	2.611	5.5	2.776	2.667	3.9
5	2.701	2.586	4.3	2.705	2.583	4.5	2.714	2.544	6.3	2.707	2.571	5.0
6	2.842	2.571	9.5	2.886	2.560	11.3	2.891	2.486	14.0	2.873	2.539	11.6
7	3.104	2.758	11.1	3.112	2.694	13.4	3.108	2.618	15.8	3.108	2.690	13.4
8	2.715	2.621	3.5	2.788	2.674	4.1	NA	NA	NA	2.752	2.648	3.8
9	2.449	2.181	10.9	2.457	2.095	14.7	2.440	2.090	14.3	2.449	2.122	13.3
10	2.458	2.268	7.7	2.442	2.254	7.7	2.454	2.210	10.3	2.455	2.244	8.6
11	2.798	2.621	6.3	2.785	2.566	7.9	2.768	2.429	13.2	2.784	2.539	9.1
12	2.766	2.731	1.3	2.804	2.584	7.8	2.782	2.575	7.4	2.784	2.630	5.5
13	2.856	2.601	8.9	2.838	2.558	9.9	2.829	2.514	8.8	2.841	2.558	9.2
14	2.823	2.490	11.8	2.811	2.474	12.0	2.841	2.445	13.9	2.825	2.470	8.6
15	2.879	2.817	2.2	2.860	2.787	2.6	2.872	2.598	9.5	2.870	2.734	4.8
16	NA	2.688	NA	NA	2.647	NA	NA	2.664	NA	NA	2.666	NA
17	2.748	2.560	6.8	2.721	2.612	4.0	2.704	2.603	3.7	2.724	2.592	3.5
18	2.762	2.690	2.6	2.714	2.642	2.7	2.722	2.647	2.8	2.733	2.660	2.7

MRD = Maximum Relative Density of a mix. VIP = Voids in Pavement (%)
BRD = Bulk Relative Density of cores. NA = Results not available.

Table 15: Permeability Results

Section	Water Permeability (ml/min.) Age (year) 0	2	5	7	Air Permeability @ 11 yr.(cm) Passing Lane	Centre Lane	Driving Lane
1	-	1.1	0	0	.imp	.imp	8.6E-12
2	-	1.7	-	0	.imp	.imp	2.2E-11
3	70	3.2	2.5	0	6.6E-10	1.3E-9	1.9E-9
4	-	1.2	-	0	.imp	.imp	.imp
5	-	1.9	-	0	6.4E-10	.imp	.imp
6	-	50	-	3	3.0E-9	6.0E-9	1.3E-9
7	-	4	0	0	10.0E-10	2.6E-10	3.2E-11
8	-	4	-	0	.imp	.imp	.imp
9	vh	30	5.5	0	1.9E-9	2.4E-9	1.5E-10
10	22	2	-	0	.imp	.imp	.imp
11	-	2.8	-	0	1.6E-9	.imp	7.1E-11
12	-	2.4	-	0	6.6E-11	3.1E-11	-.-----
13	vh	78	20	7	1.3E-9	1.3E-10	-.-----
14	vh	150	-	13.5	6.6E-9	1.2E-9	1.0E-8
15	6	3	-	0	6.7E-12	.imp	.imp
16	22	2.5	-	0	1.4E-10	.imp	.imp
17	-	4.3	-	0	.imp	.imp	1.0E-9
18	-	2.4	-	0	3.4E-11	.imp	.imp

- = results not available, vh = very high permeability,
imp = impermeable, E = x10^{-}

Table 16: Properties of Recovered Asphalt Cement (at 11 yrs)

Test Mix No.	PENETRATION 25 C			PENETRATION 4 C			VISCOSITY Kinem.,135C(mm^2/s)			VISCOSITY Absolute,60C(Pa*s)			SOFTENING POINT R & B (C)		
	DL	CL	PL	DL	CL	PL	DL	CL	PL	DL	CL	PL	DL	CL	PL
1	47	50	41	10	9	4	669	646	787	764	637	1251	55.7	58.1	58.5
2	48	56	38	9	9	9	628	565	826	640	469	1393	54.2	54.3	54.5
3	16	17	18	3	3	4	2532	2157	2718	13655	13714	17988	72.9	69.8	73.2
4	40	34	37	7	7	7	759	942	813	1028	1928	1502	57.4	60.9	59.2
5	26	34	30	3	7	6.5	1376	893	1020	3600	1428	2321	65.8	61.4	62.4
6	21	22	17	5	6	4	2159	1350	2671	13024	15231	20088	72.8	73.0	75.0
7	23	26	26	7	8	9	1321	1170	1251	5877	3395	4381	65.8	65.0	64.6
8	41	40	NA	7	7.5	NA	619	577	NA	668	561	NA	55.5	54.8	NA
9	17	14	16	2.5	3	3	1272	1892	1626	4510	9347	8641	64.7	69.3	66.0
10	28	28	33	7	8	11	841	778	1089	1839	1371	2164	59.7	58.5	61.2
11	29	26	24	6	5	7	871	917	1025	1574	2095	2867	60.1	61.0	62.7
12	27	26	49	5	6	9	927	1072	573	1840	2221	399	61.9	61.7	52.7
13	21	21	17	7	7	4	1152	1006	1707	2936	2186	5102	62.3	61.9	65.3
14	20	13	14	4	3	4	1303	2118	1897	5627	11089	9629	64.4	69.2	69.0
15	29	29	23	5	7	4	765	692	915	1158	1145	2338	58.0	58.2	61.4
16	21	23	30	5	5	6	1138	991	563	3302	2442	1081	64.1	62.5	57.9
17	26	28	35	4	8	7	2107	1890	1317	14435	7941	3246	69.5	67.3	62.6
18	53	49	44	9	8	8	529	537	617	396	441	570	51.8	51.9	52.3

* NA = Result not available.
* Original A.C. grade used was 85/100 pen. for all mixes except for mix No. 17 where 60/70 pen. grade was used.

Table 17

STRENGTH CHARACTERISTICS OF THE EXPERIMENTAL MIXES AT 11 VPS

Test mix No.	MARSHALL TEST						INDIRECT TENSILE TEST			RESILIENT MODULUS TEST			CORE THICKNESS		
	STABILITY (kN)			STIFFNESS RATIO (N/mm)			TENSILE STRENGTH (kPa)			MODULUS Mr (MPa)			(mm)		
	D.L.	C.L.	P.L.	D.L.	C.L.	P.L.	D.L.	C.L.	P.L.	D.L.	C.L.	P.L.	D.L.	C.L.	P.L.
1	12.9	14.45	17.45	4e23	3823	5541	985	646	1048	2985	2500	2839	36.5	38	37
2	11.64	17.47	1e.87	3399	3882	4027	713	889	1003	2762	3178	3339	39	37	31
3	18.92	20.16	2e.2e	4802	4130	6777	1173	1034	1058	4260	45e8	4389	39.5	41	44
4	15.38	17.85	17.93	3790	4113	4102	736	892	NA	2614	3300	2834	50	38	37
5	11.54	15.01	12.68	1800	2300	1900	1162	833	798	3653	5394	4961	45	34	3e
6	14.82	20.11	2e.42	2300	38e0	e706	682	e42	515	2251	1993	2864	43	38	33
7	22.88	22.78	21.86	5332	5210	5096	728	650	e43	3253	2939	31e1	31	44	40.5
8	21.3	1e.36	NA	682e	5330	NA	1027	941	NA	2899	2252	NA	47.5	48	NA
9	29.12	24.38	2e.13	5e91	4175	5144	987	826	865	4915	2009	3400	33	51	50
10	19.99	18.01	22.1e	e247	4955	510e	759	851	e27	2492	2020	2203	45	48	3e
11	14.83	13.33	18.42	2300	2000	2900	938	813	738	2971	3127	2e07	33	35	34
12	10.14	15.61	18.05	1500	2400	2800	NA	1035	851	NA	4022	3e52	NA	31	30
13	14.91	12.12	NA	2300	1900	NA	882	e59	941	3099	2022	3902	34	33	35
14	17.75	23.45	23.2e	5298	8783	9745	e97	830	NA	2912	NA	NA	38.5	42	NA
15	13.5	13.76	19.17	2860	2100	342e	624	675	959	2043	2297	375e	28.5	32	25
16	NA	NA	NA	NA	NA	NA	1001	119e	1147	4453	141e	3206	29	40	35
17	14.8	11.1	10.41	2300	1700	1600	715	672	7e4	2277	1572	2401	39	47	52
18	15.93	15.4	17.84	4e15	4557	5083	540	NA	994	2222	NA	210e	40	NA	34

LEGEND:

D.L.= Driving Lane

C.L.= Centre Lane

P.L.= Passing Lane

NOTES:

1.Recompacted specimens were used for the Marshall test.

2.Resilient Modulus Test and Indirect Tensile Test were carried out using the same field cores taken from the right-wheel track. Both tests were performed at room temperature (i.e. approx. 23 C).

Fig. 1

Traffic Volume (AADT) on Hwy 401 Toronto By-pass between Dufferin and Keele Street

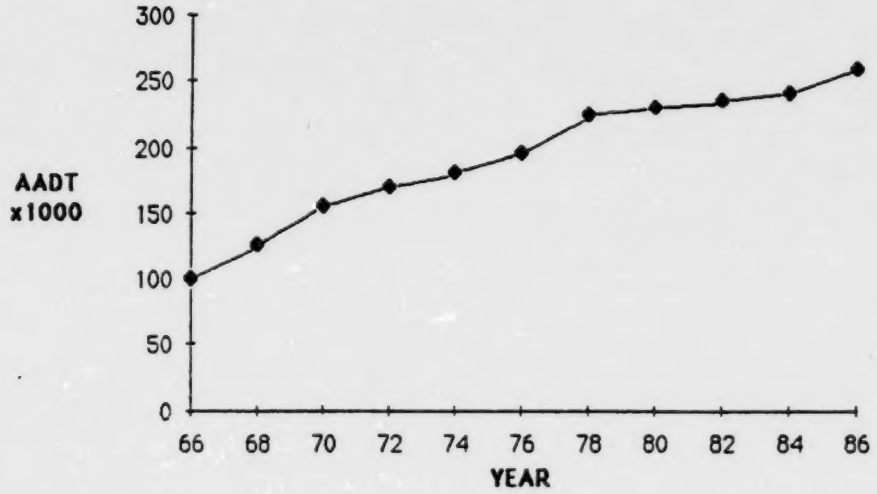
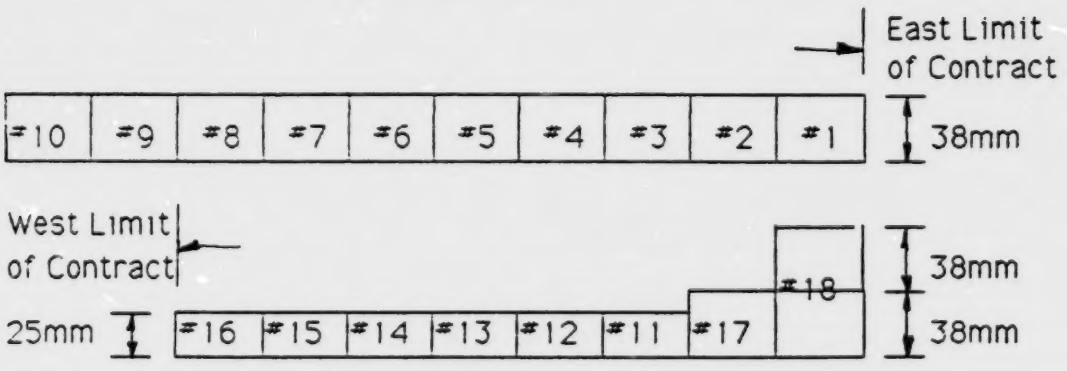


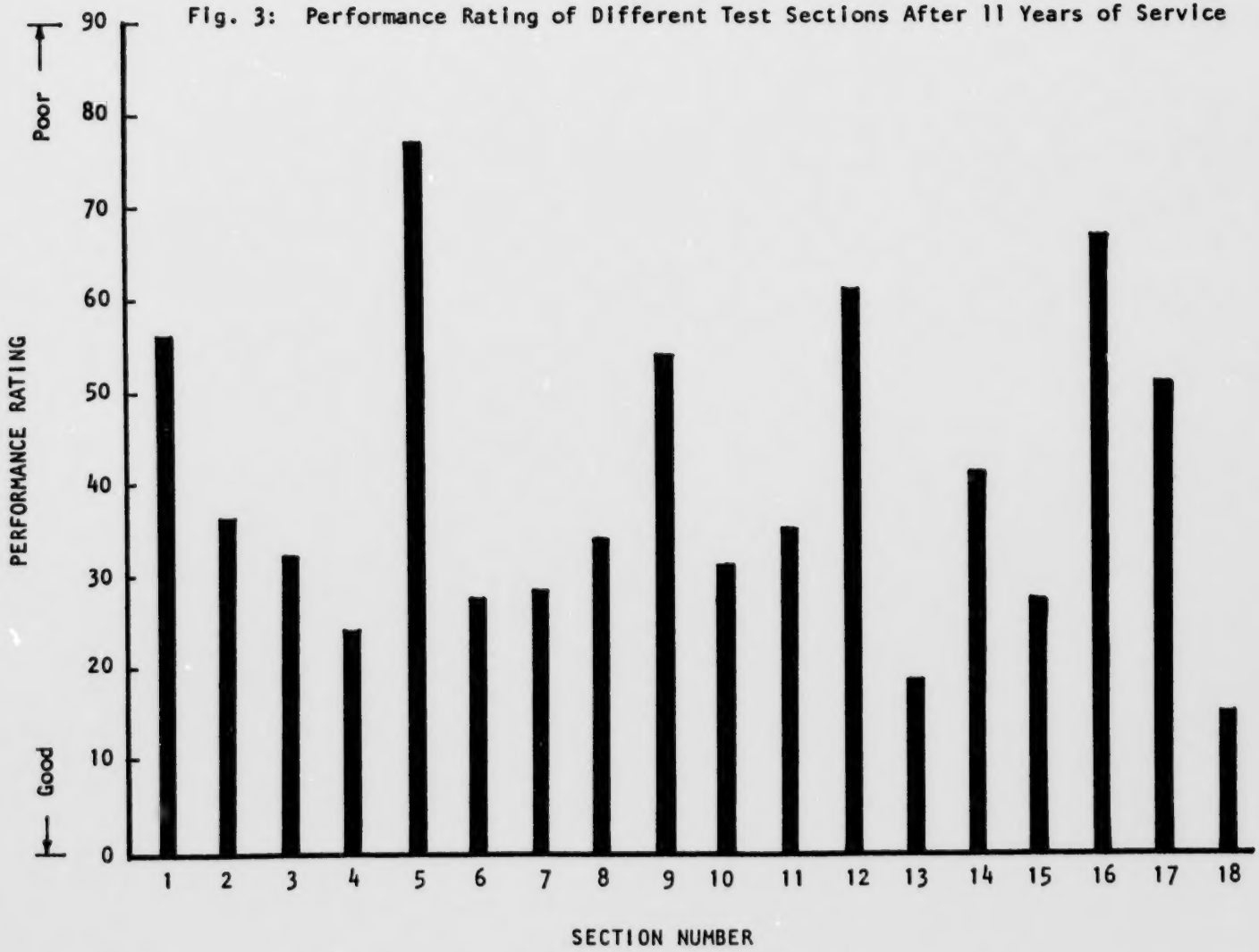
Fig. 2

LAYOUT AND THICKNESS OF THE 18 TEST SECTIONS



Note :
1) Numbers shown are section numbers
2) Each Section is 137 m long

Fig. 3: Performance Rating of Different Test Sections After 11 Years of Service



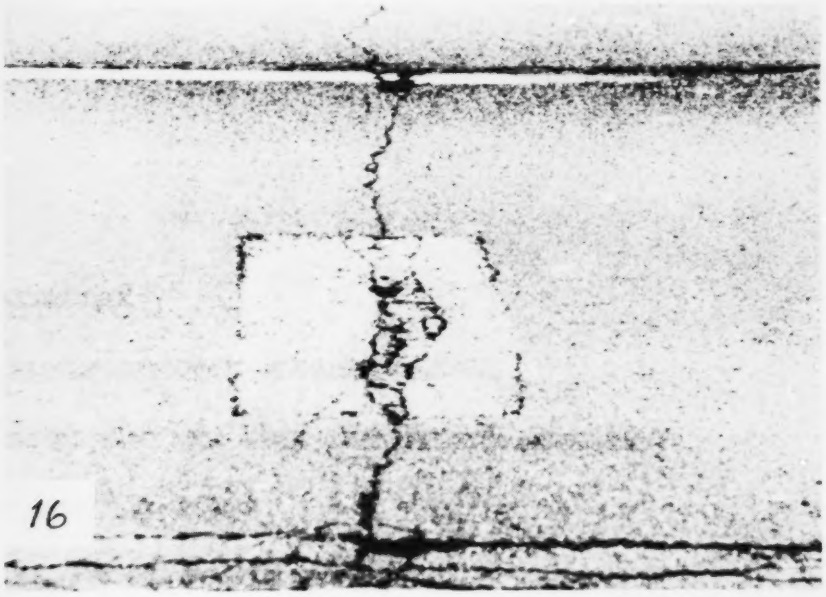


Fig. 4: Reflection Cracking Through the Repaired Area (Section 16)

Fig. 5: Longitudinal Joint Treatment and Patching on Ravelled Area (Section 3)

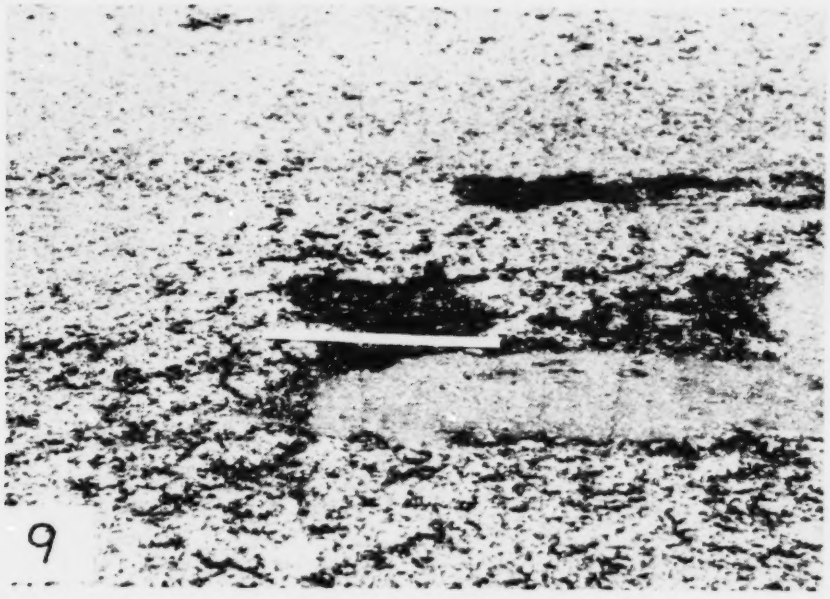


Fig. 6: Ravelling Along the Right Wheel Track in the Driving Lane (Section 9)

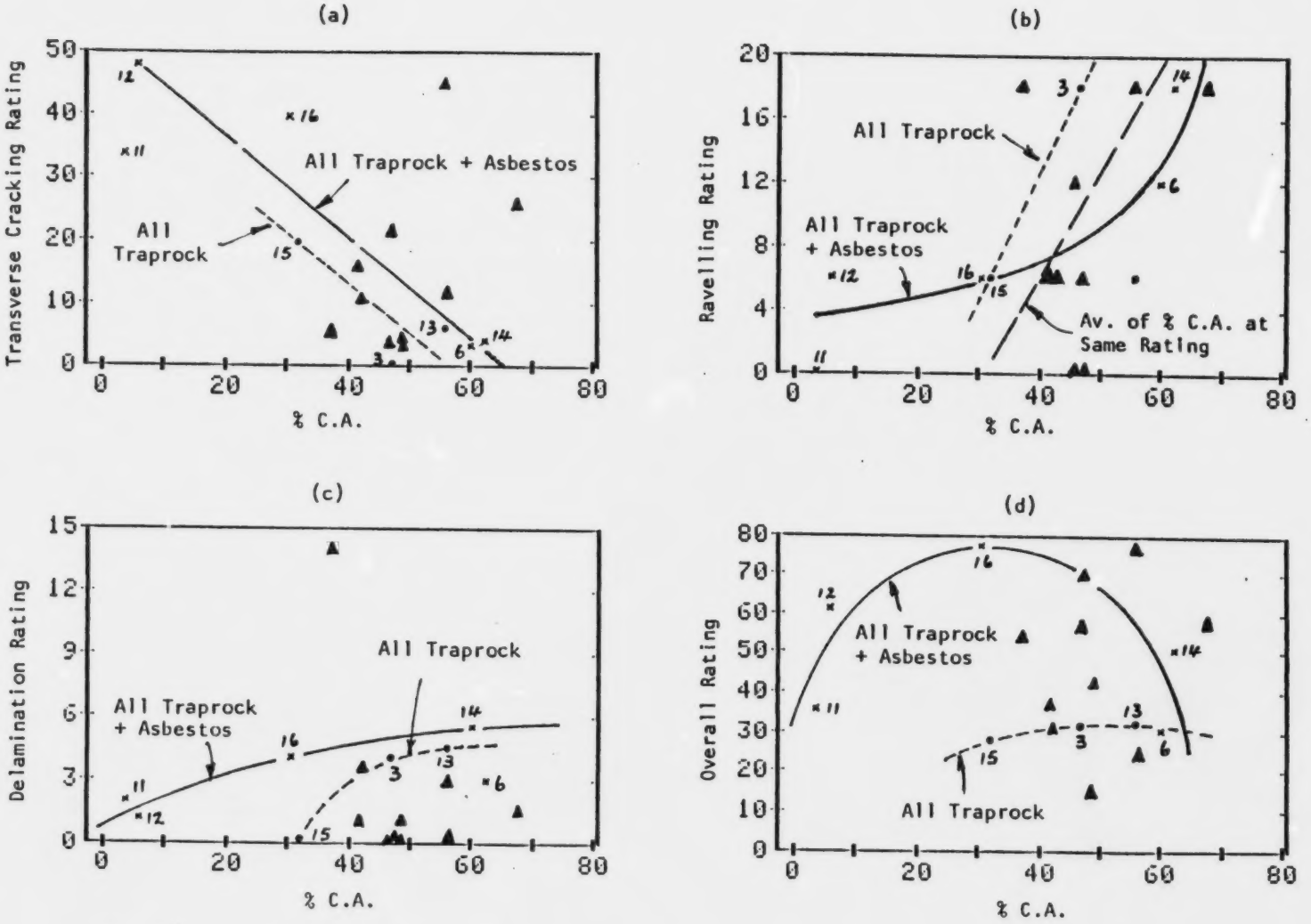


Fig. 7: Influence of Coarse Aggregate Content (% C.A.) on the Different Performance Parameters (number = section #; ▲ = other mixes)

Fig. 8: Patching on the Delaminated Area (Section 14)

Fig. 9: Wheel Track Rutting Along the Driving Lane in Section 2

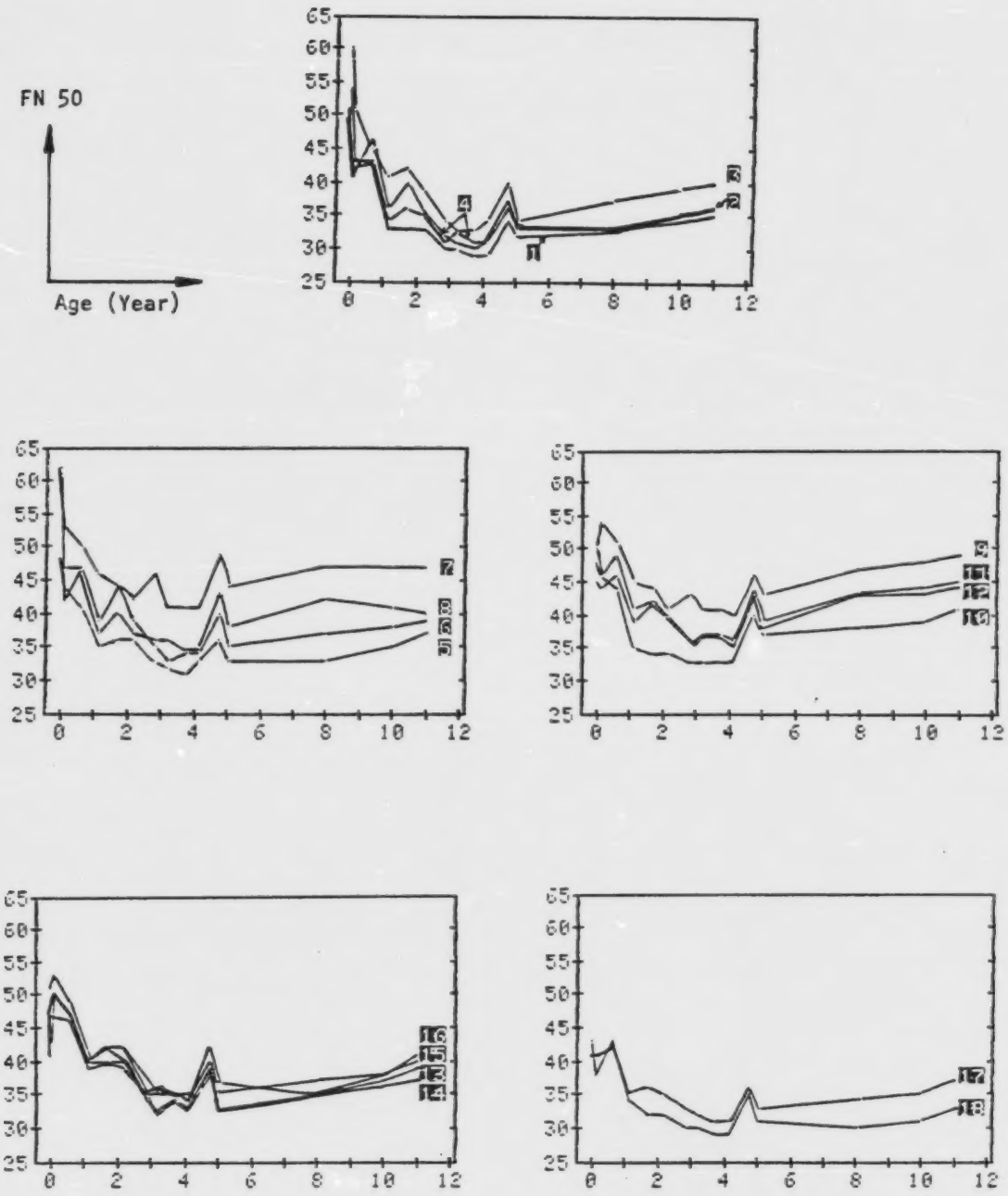


Fig. 10: Friction Number (FN) at 50 km/h at Different Age of the Pavement in the Driving Lane

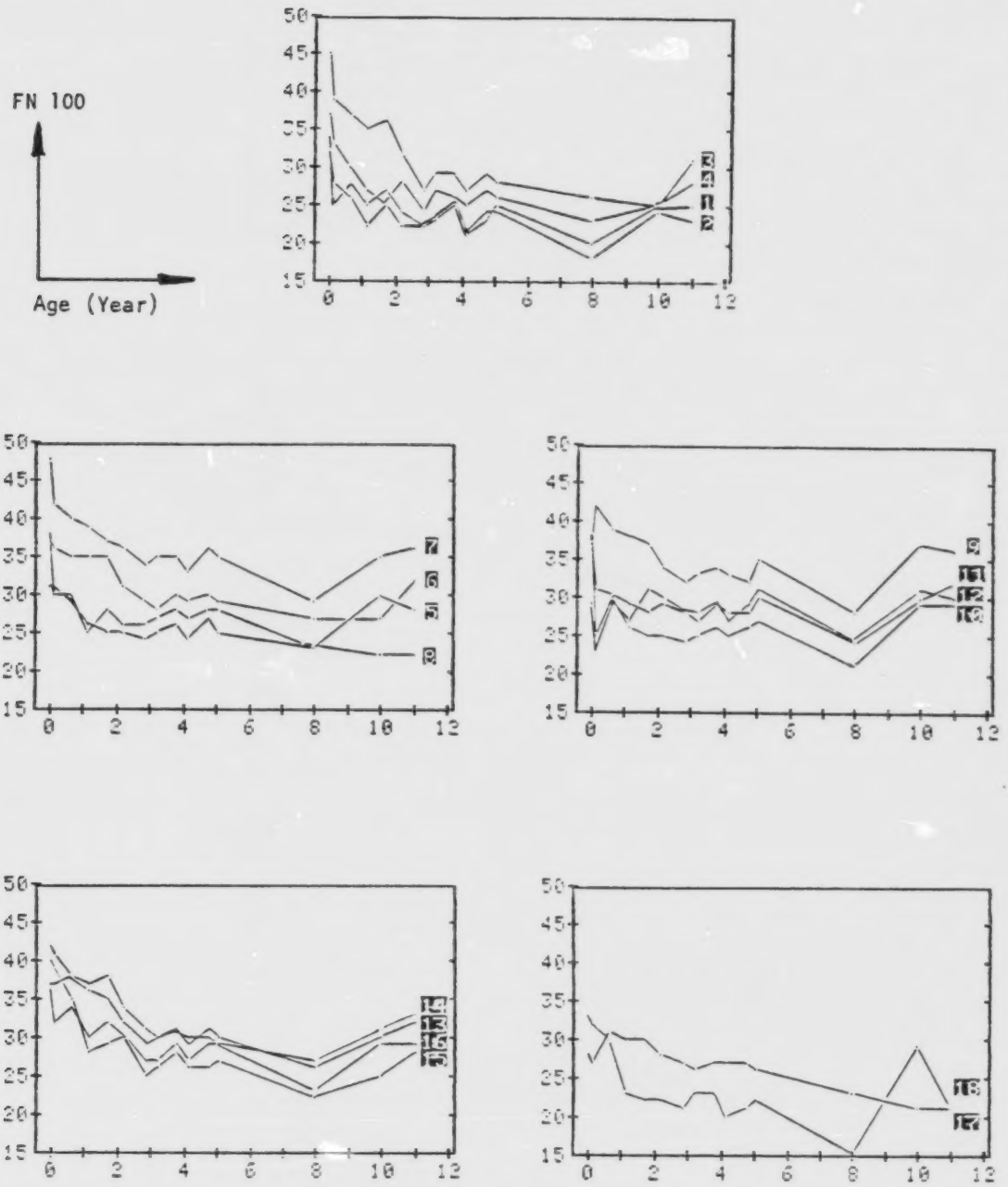


Fig. 11: Friction Number (FN) at 100 km/h at Different Age of the Pavement in the Driving Lane

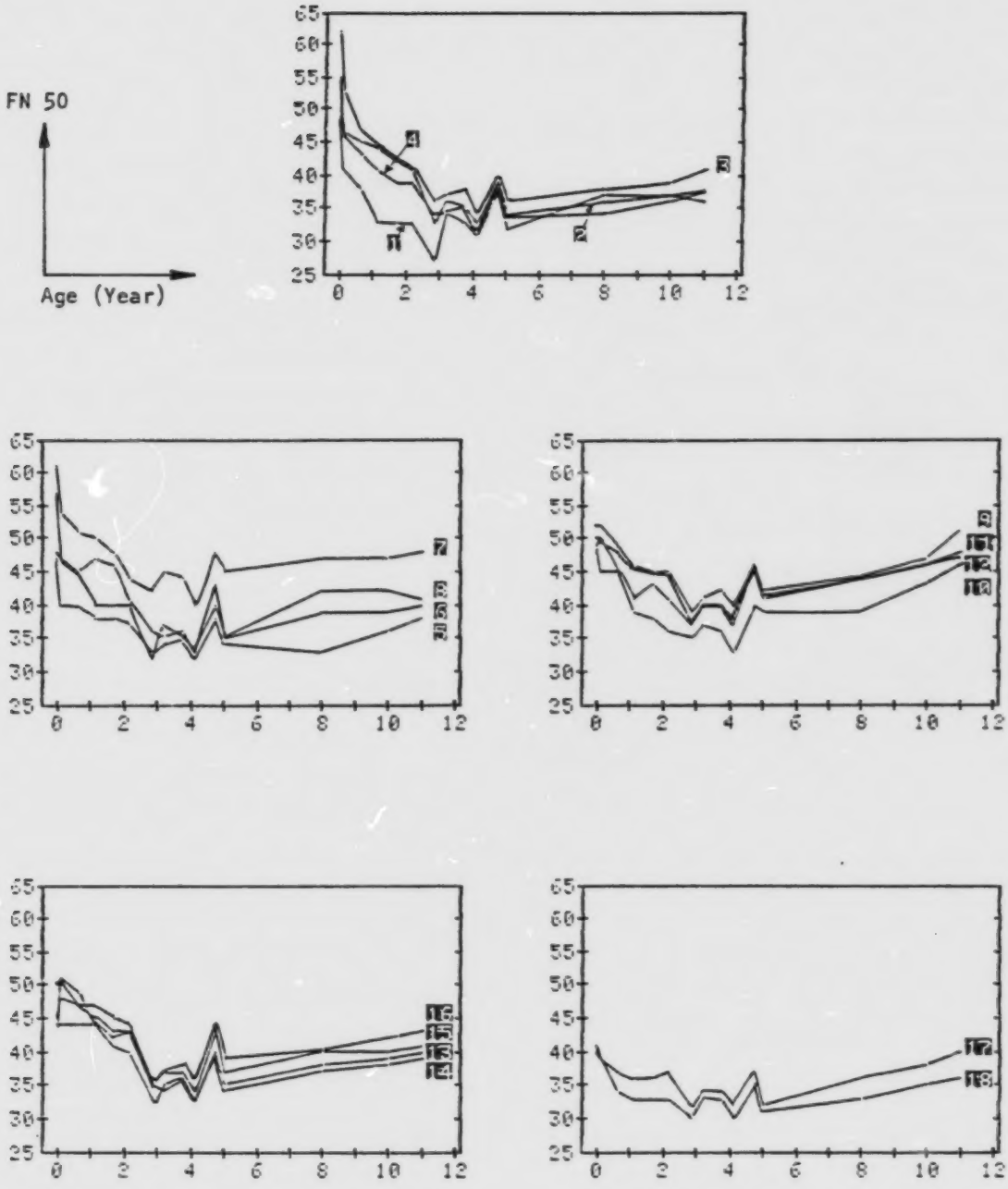


Fig. 12: Friction Number (FN) at 50 km/h at Different Age of the Pavement in the Centre Lane

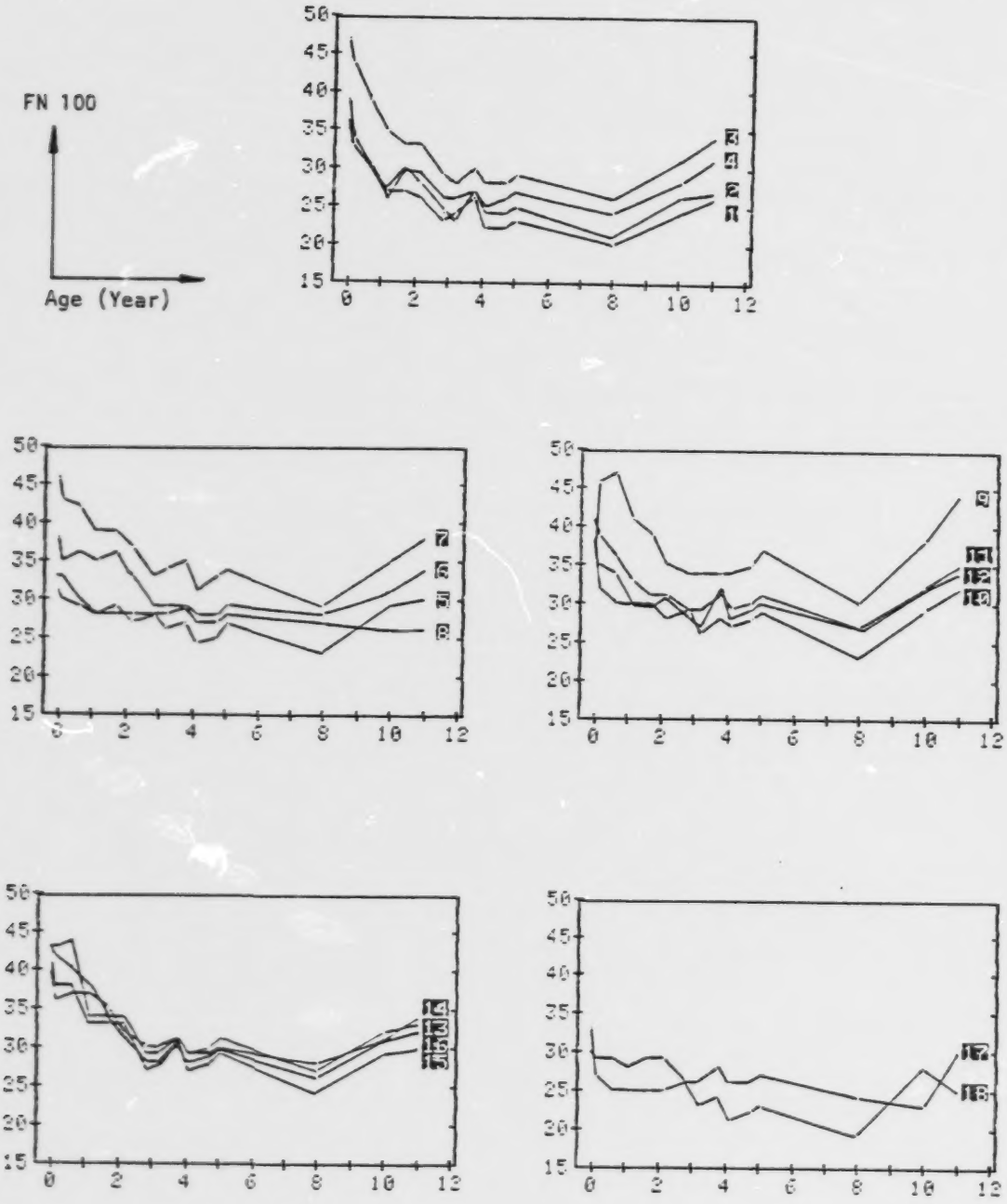


Fig. 13: Friction Number (FN) at 100 km/h at Different Age of the Pavement in the Centre Lane

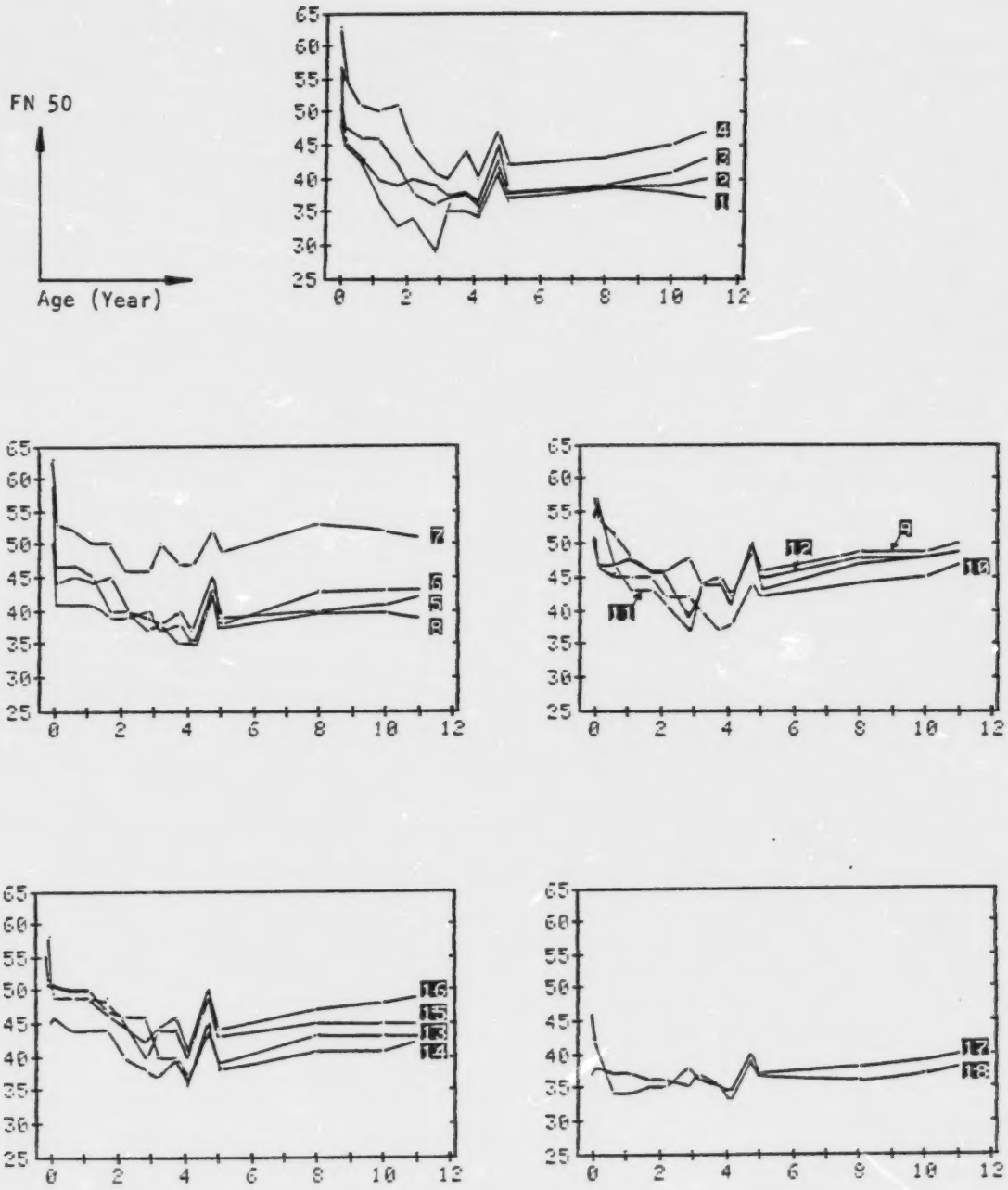


Fig. 14: Friction Number (FN) at 50 km/h at Different Age of the Pavement in the Passing Lane

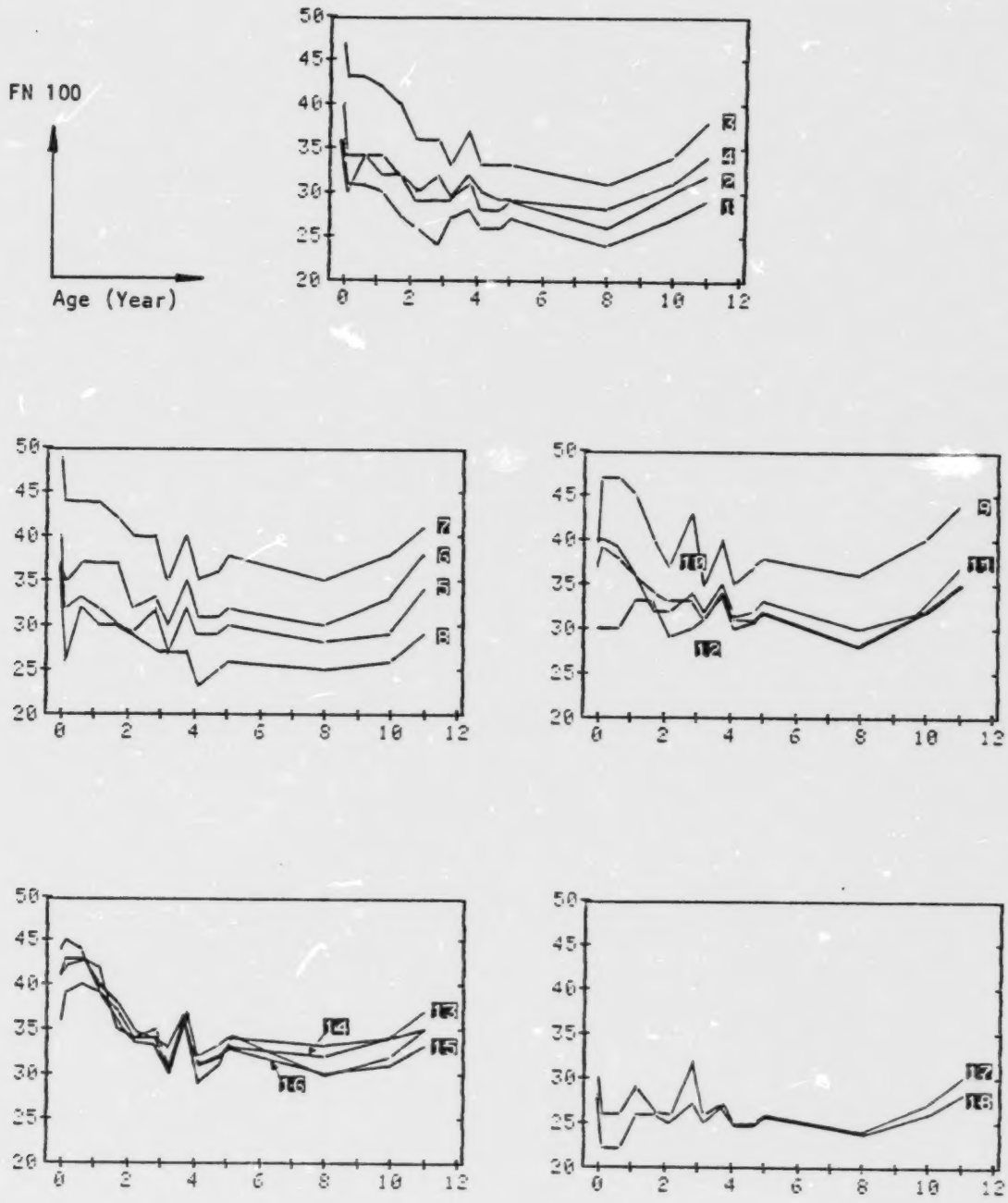


Fig. 15: Friction Number (FN) at 100 km/h at Different Age of the Pavement in the Passing Lane

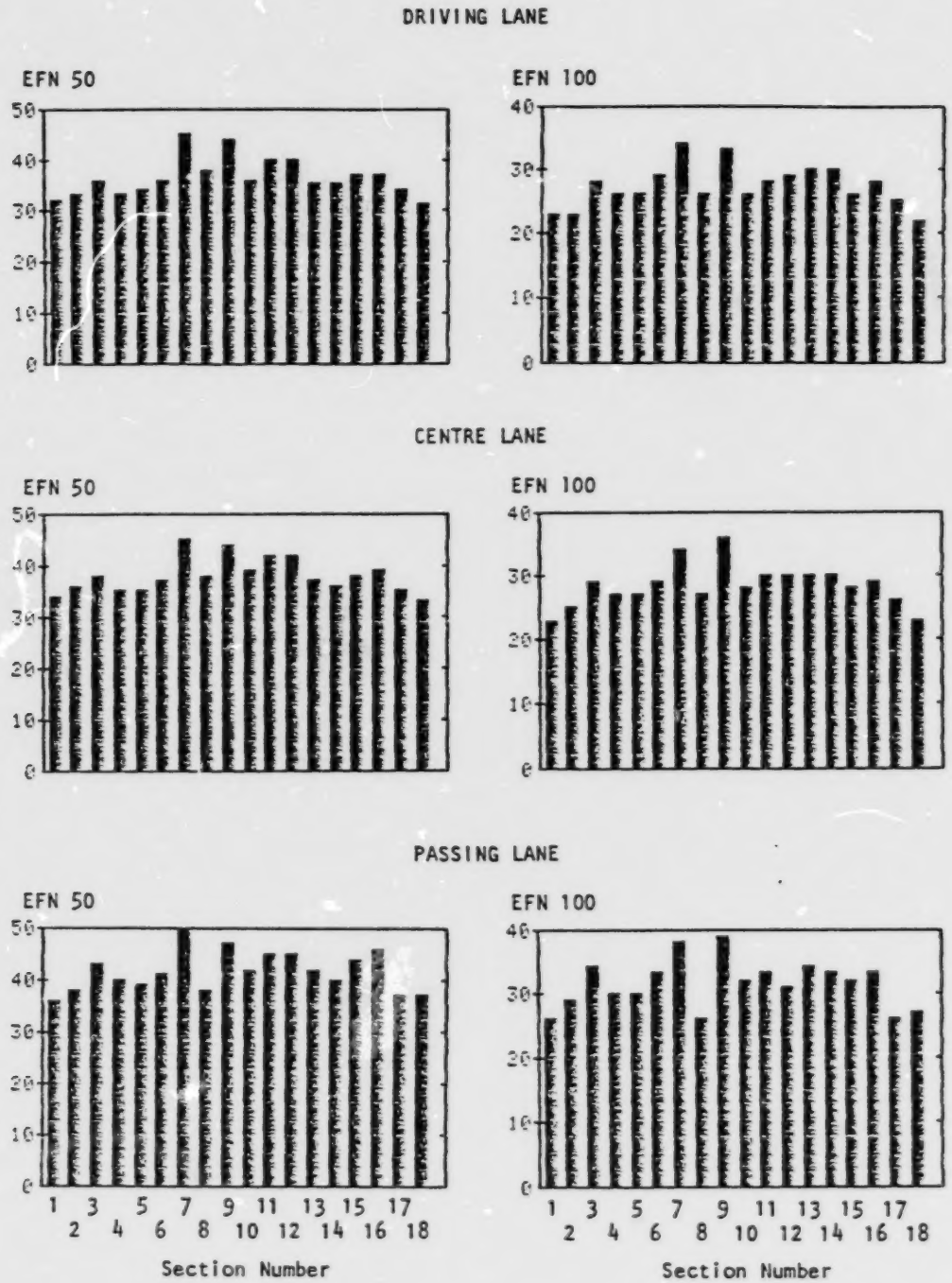

DRIVING LANE
EFN 50
EFN 100
CENTRE LANE
EFN 50
EFN 100
PASSING LANE
EFN 50
EFN 100
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
Section Number
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
Section Number

Fig. 16: Equilibrium Friction Number (EFN) Values at 50 and 100 km/h of the 18 Test Sections

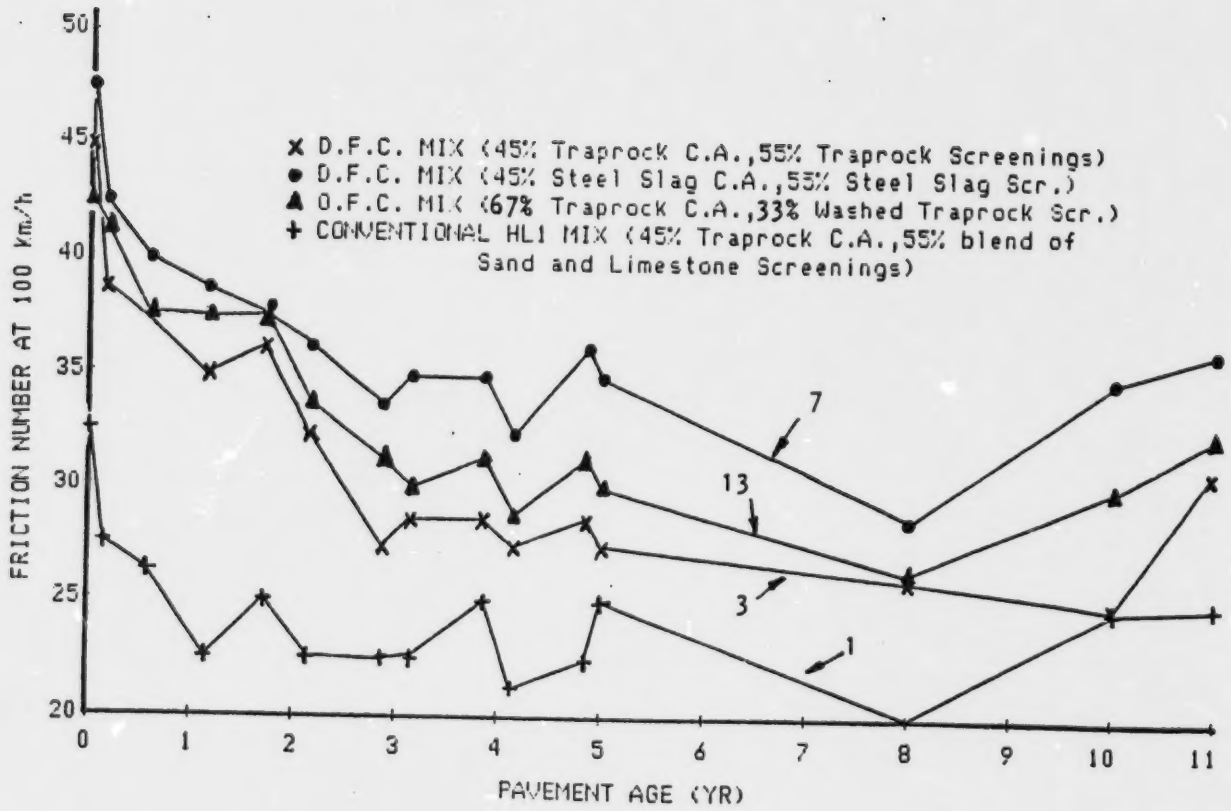


FIG. 17: CHANGE IN FRICTION NUMBER WITH TIME FOR SELECTED TEST SECTIONS

Fig. 18 Close-up of Test Section No. 7, Wheel Path, Driving Lane, After 11 Years Service

Friction Excellent, Good Macrotexture and Microtexture

Steel Slag Coarse Aggregate
Steel Slag Screenings Fine Aggregate

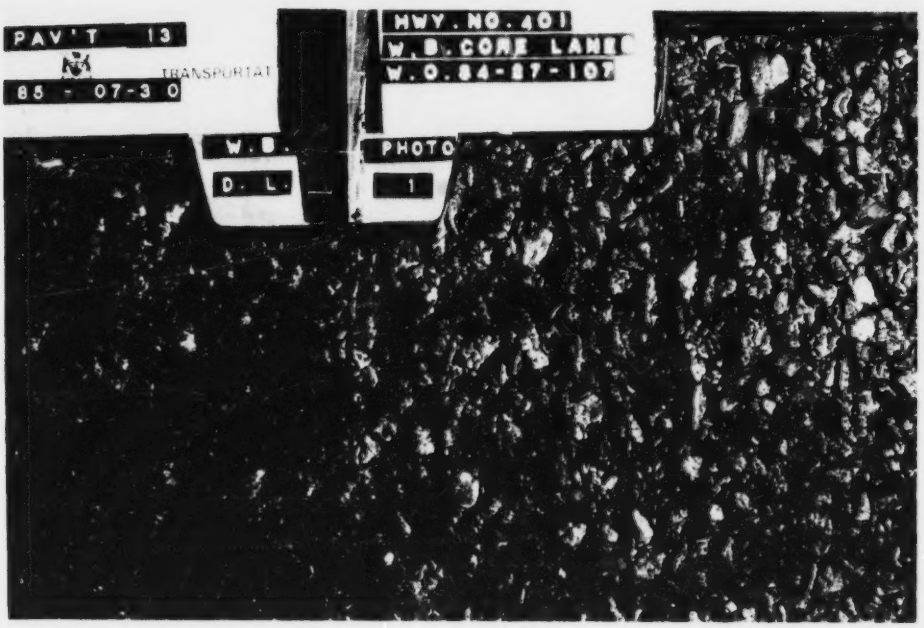


Fig. 19 Close-up of Test Section No. 13, Wheel Path, Driving Lane, After 11 Years Service

Friction Excellent,
Well Developed Macrotexture

Traprock Stone Coarse Aggregate
Traprock Screenings Fine Aggregate

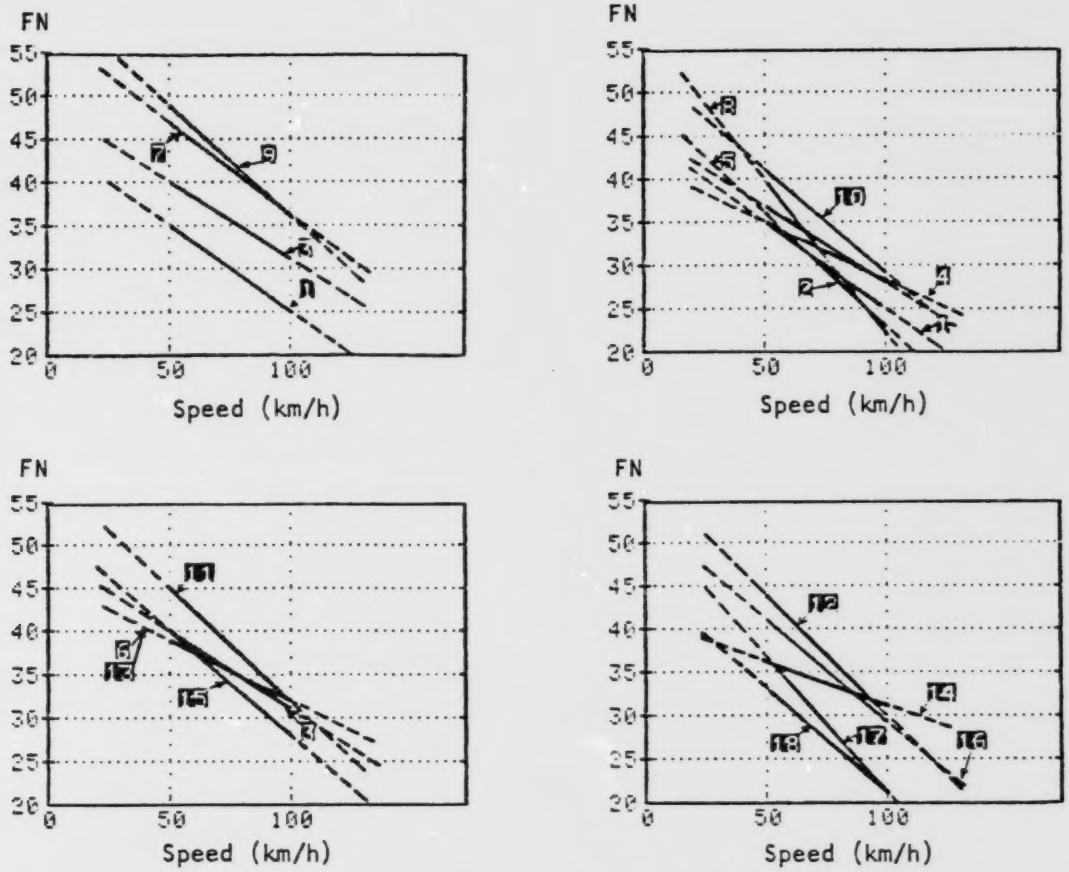


Fig. 20: Influence of Speed on the Friction Number (FN) for Different Mixes with Various Degrees of Macrotexture in the Driving Lane (After 11 Years of Service)

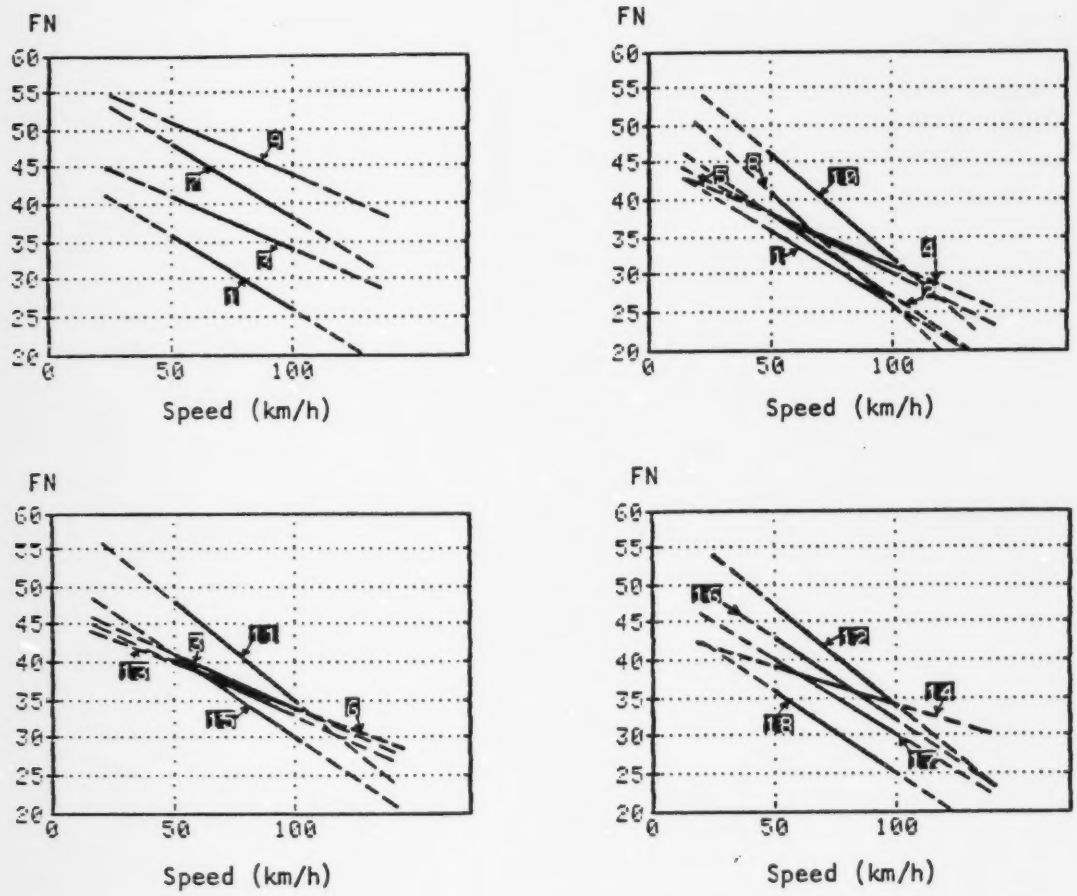


Fig. 21: Influence of Speed on the Friction Number (FN) for Different Mixes with Various Degrees of Macrotexture in the Centre Lane (After 11 Years of Service)

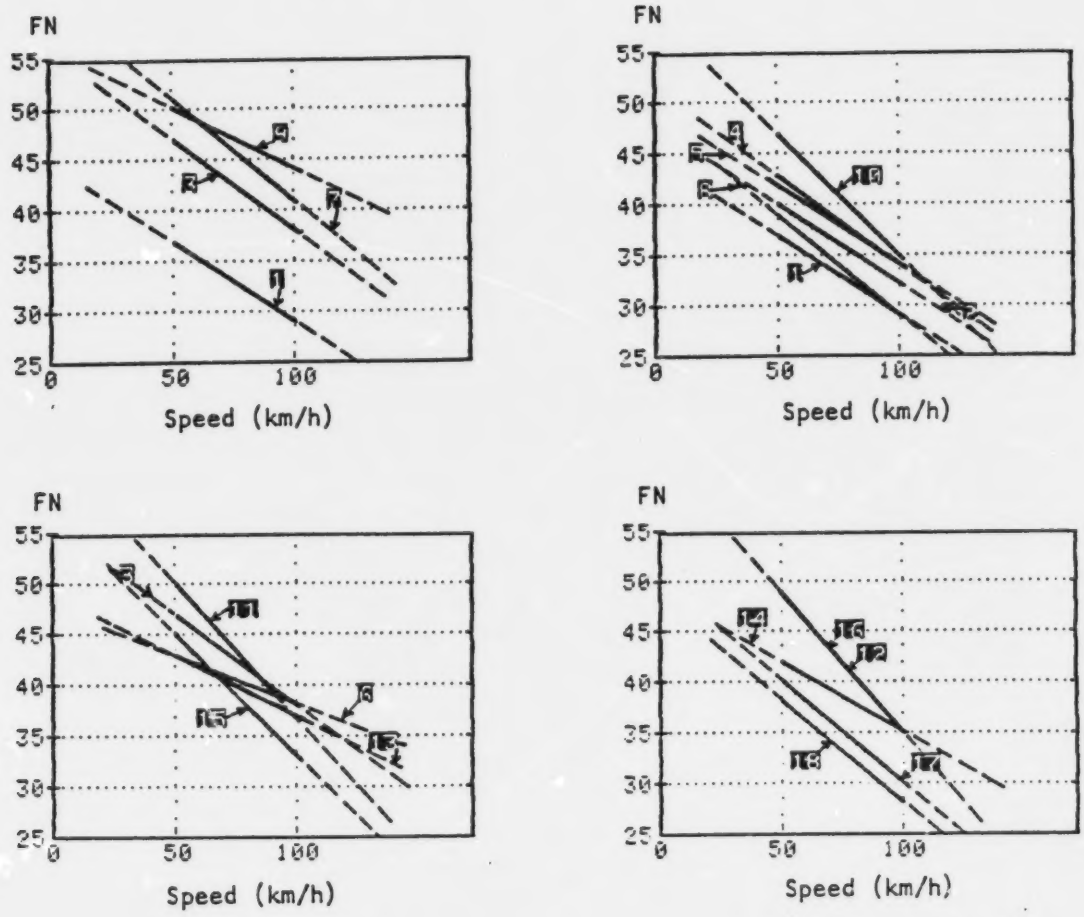


Fig. 22: Influence of Speed on the Friction Number (FN) for Different Mixes with Various Degrees of Macrotexture in the Passing Lane (After 11 Years of Service)

Fig. 23: Surface Texture of Section 1, Driving Lane

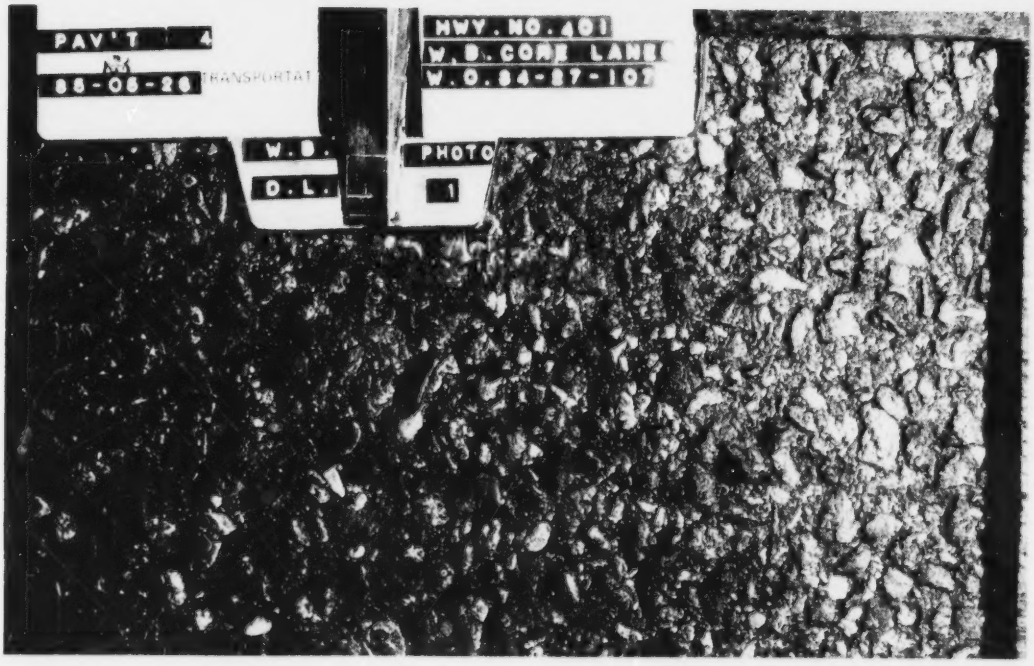


Fig. 24: Surface Texture of Section 4, Driving Lane
(Note more open than Section 1)

Fig. 25: Surface Texture of Section 9 Blast Furnace Slag Mix

Fig. 26: Surface Texture of Section 11 Sand Mix (Note the fine macrotexture)

Fig. 27: Coarse Macrotexture of Mastic Mix (Section 17)

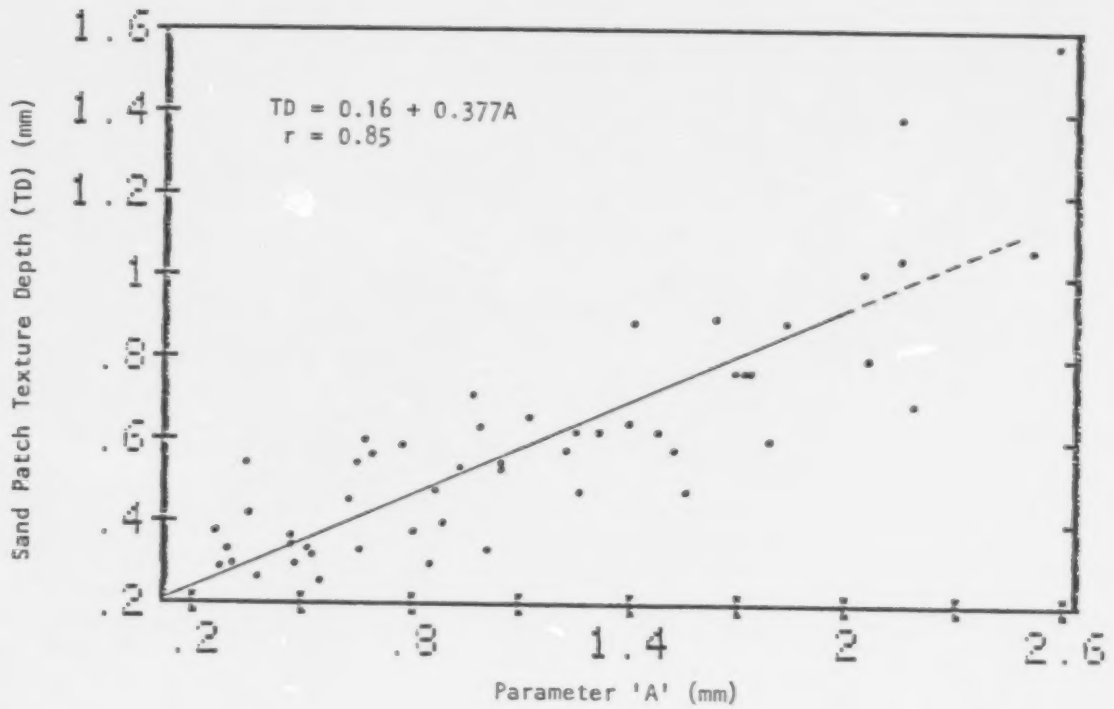


Fig. 28: Relationship Between Texture Depth by Sand Patch Method and Parameter 'A' by Photointerpretation Method

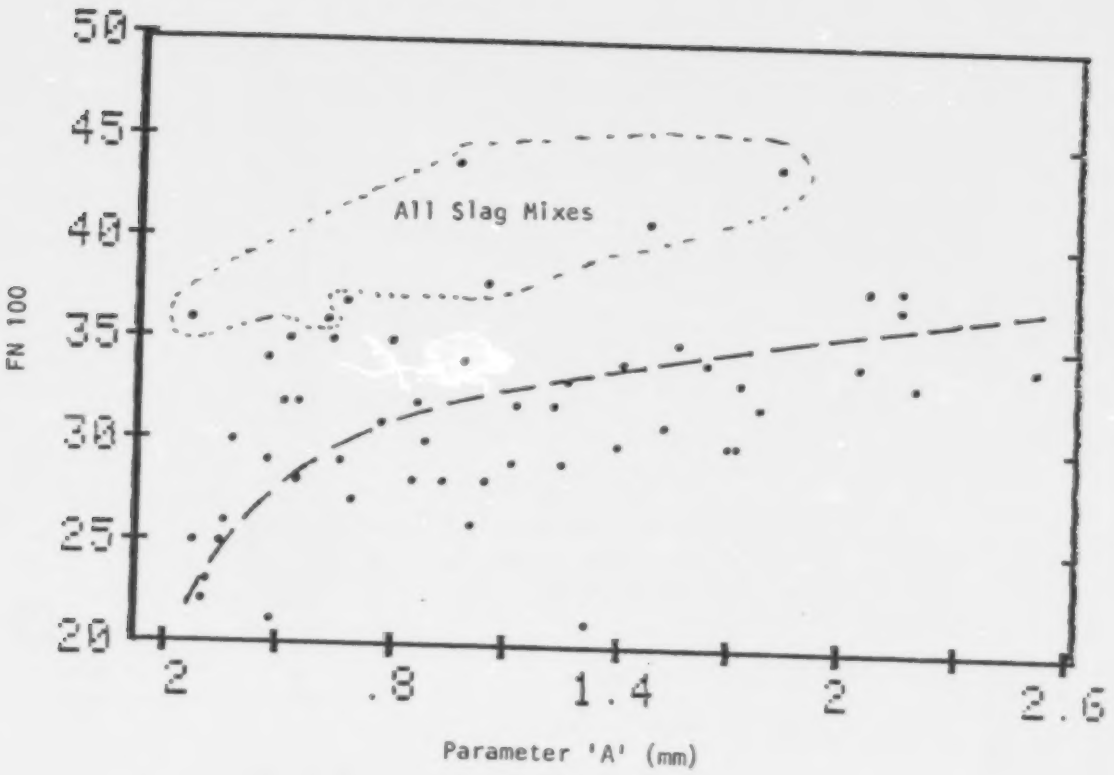


Fig. 29: Relationship Between Friction Number (FN) at 100 km/h and Parameter 'A' Measured by Photointerpretation Method

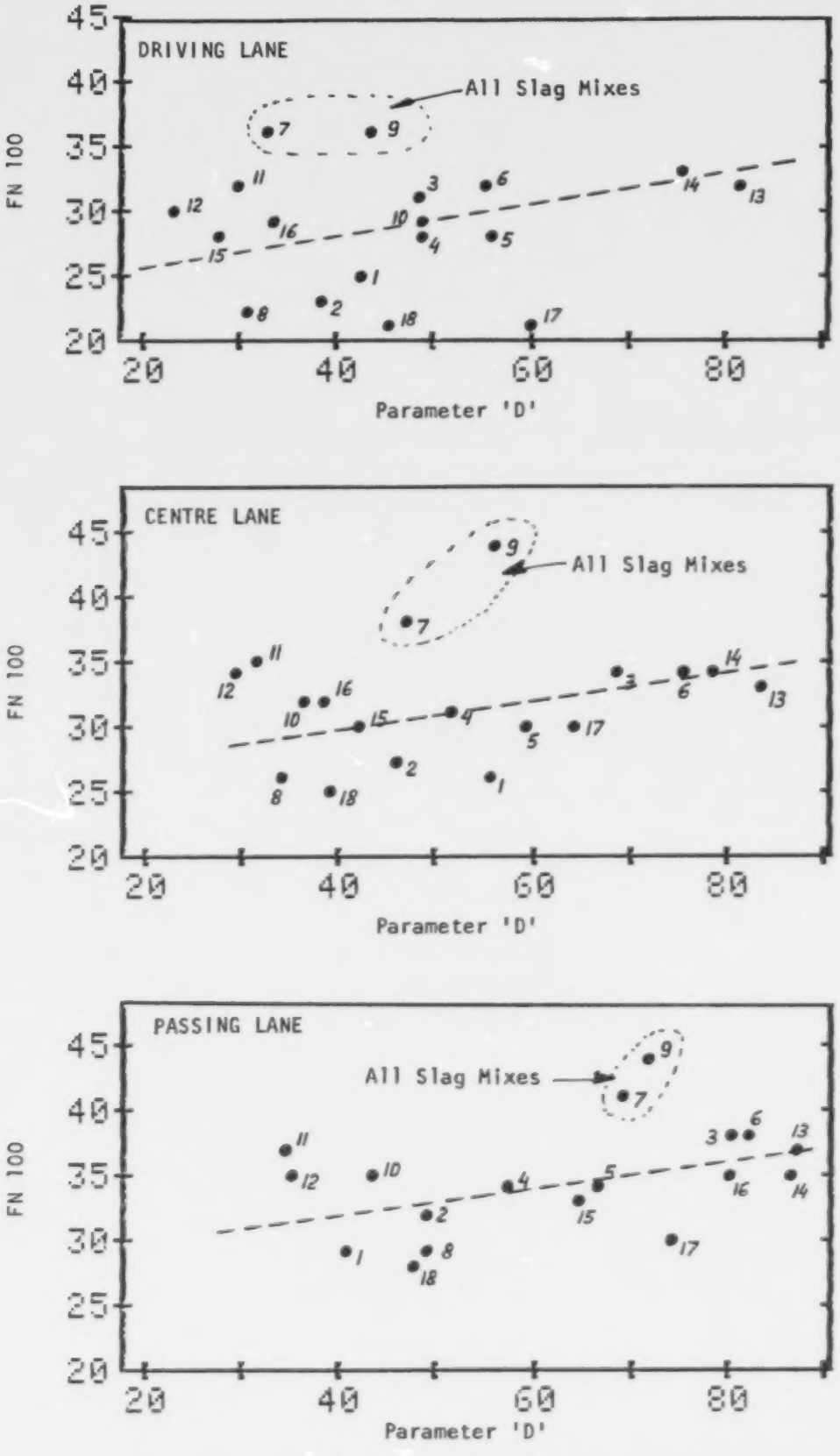


Fig. 30: Relationship of Friction Number (FN) at 100 km/h and Parameter 'D' by Photointerpretation Method for the Driving, Centre and Passing Lanes (after 11 years of service).

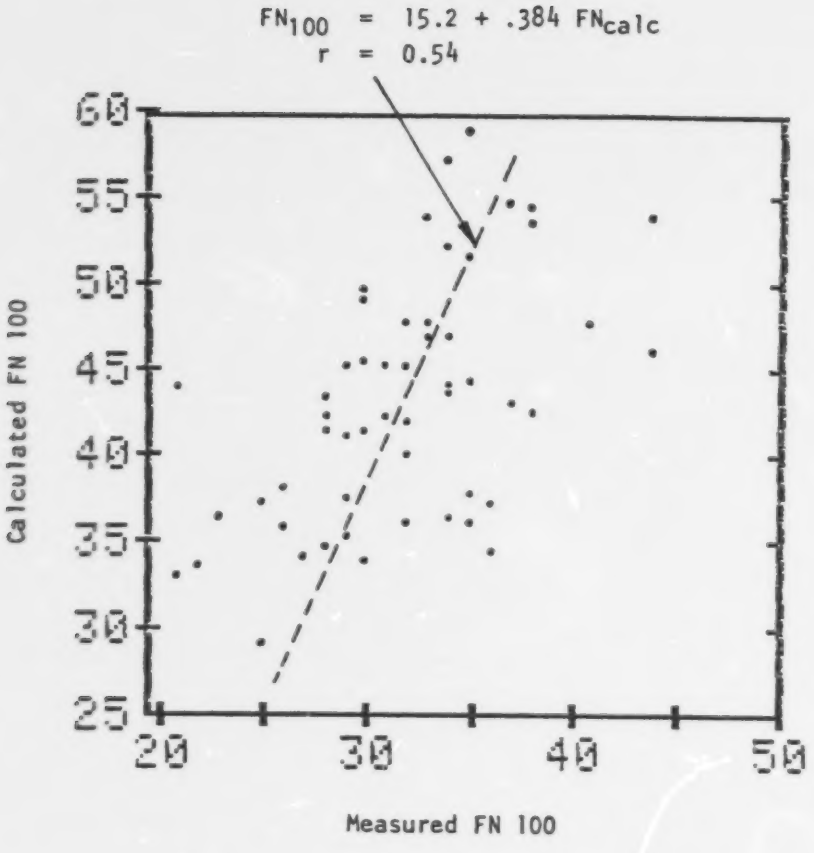


Fig. 31: Relationship Between Calculated Friction Number (FN) by Using Photointerpretation Parameters (6) and Measured FN at 100 km/h

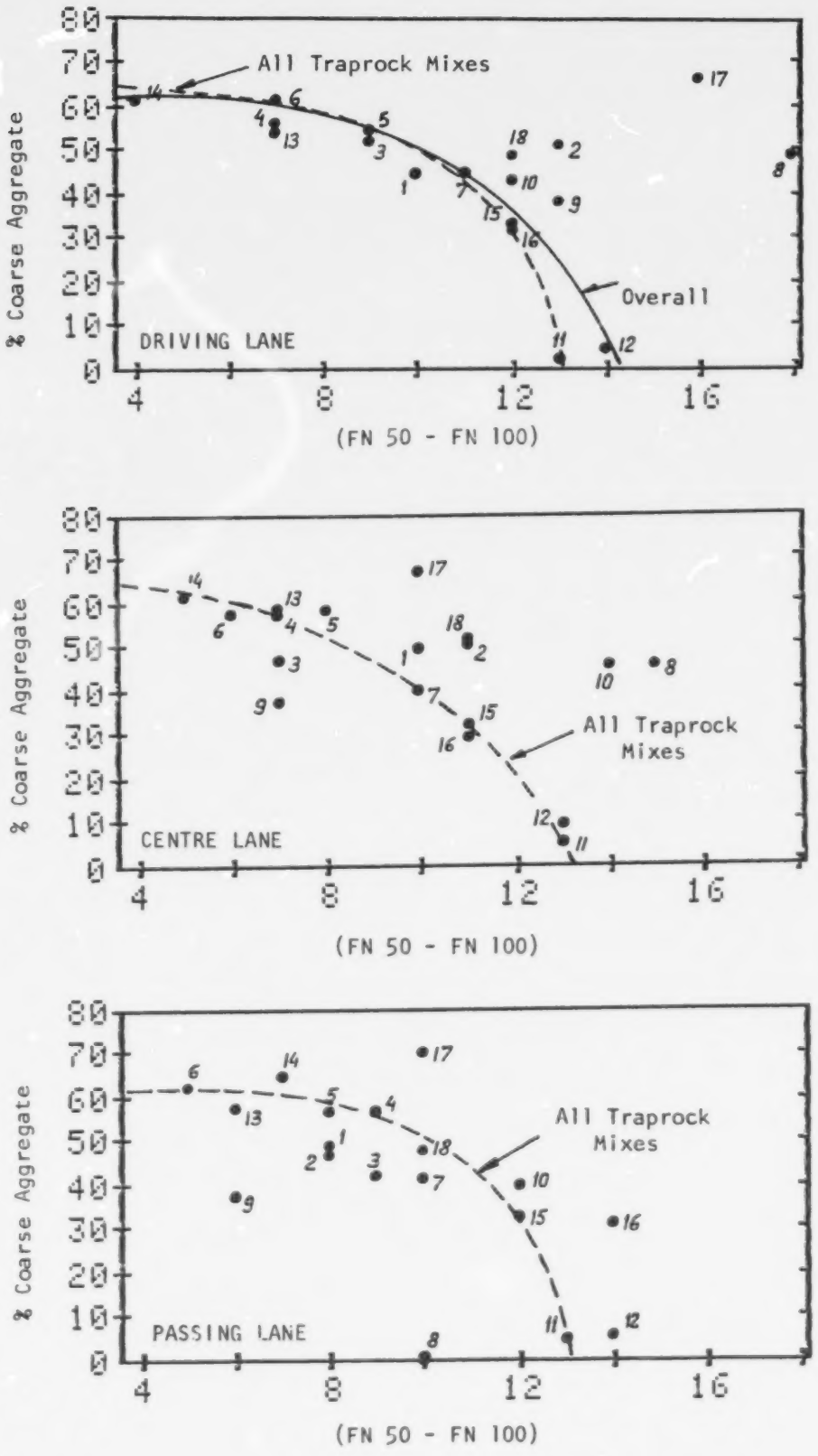


Fig. 32: Relationship of Coarse Aggregate Content (% C.A.) and Difference in Friction Number at 50 and 100 km/h

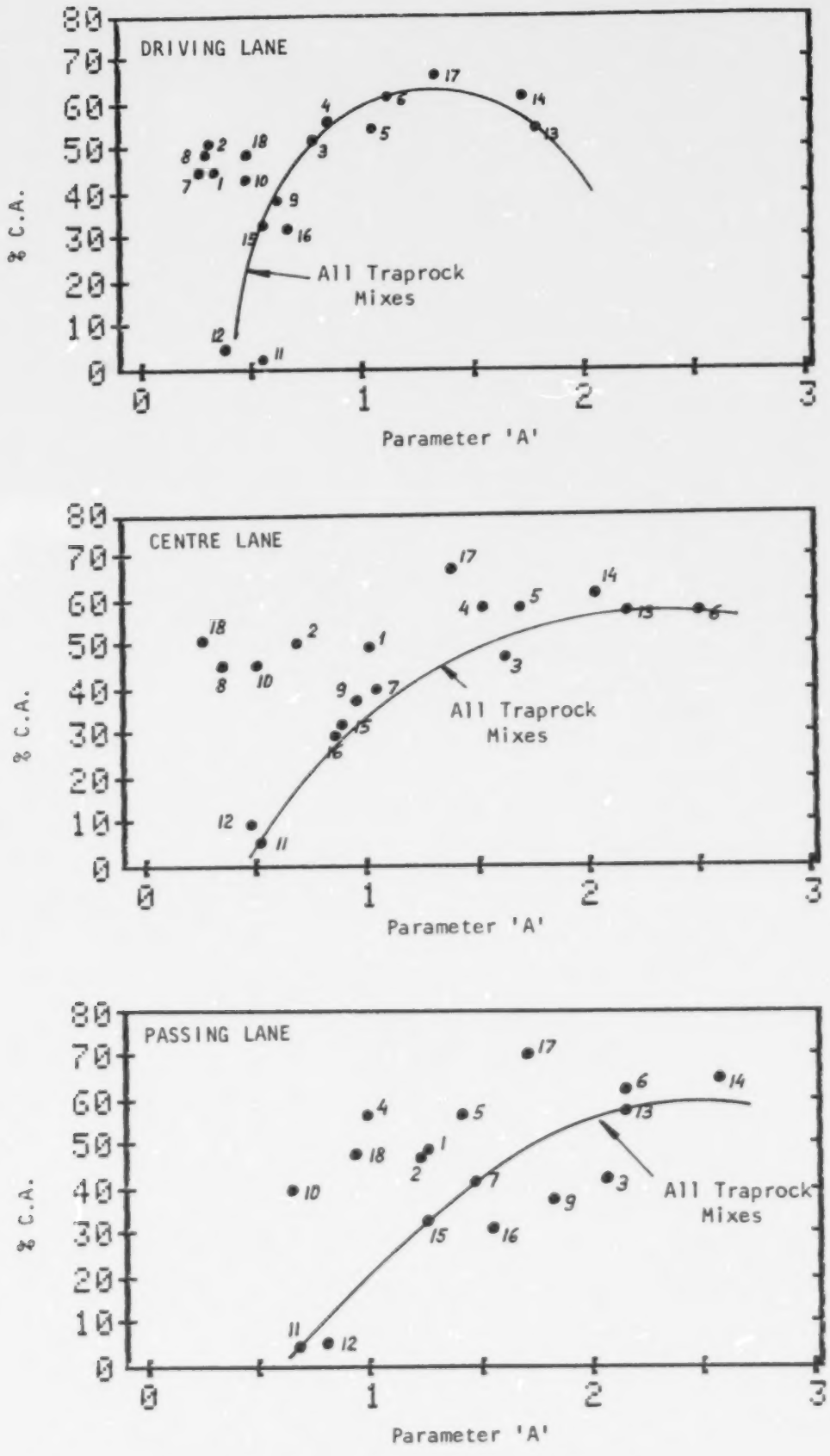


Fig. 33: Relationship Between Coarse Aggregate Content (% C.A.) and Parameter 'A' by Photointerpretation Method for the Driving, Centre and Passing Lanes

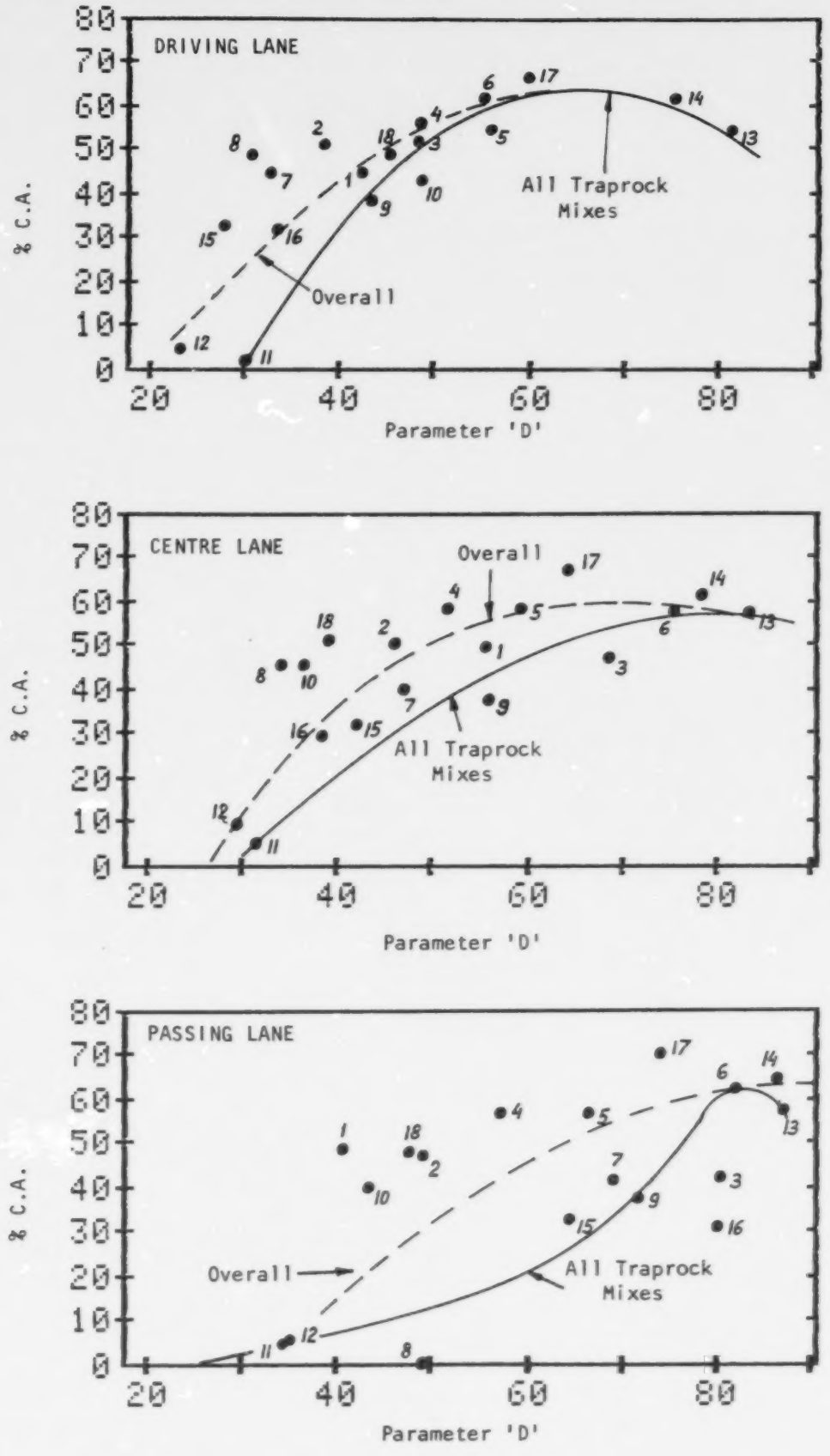
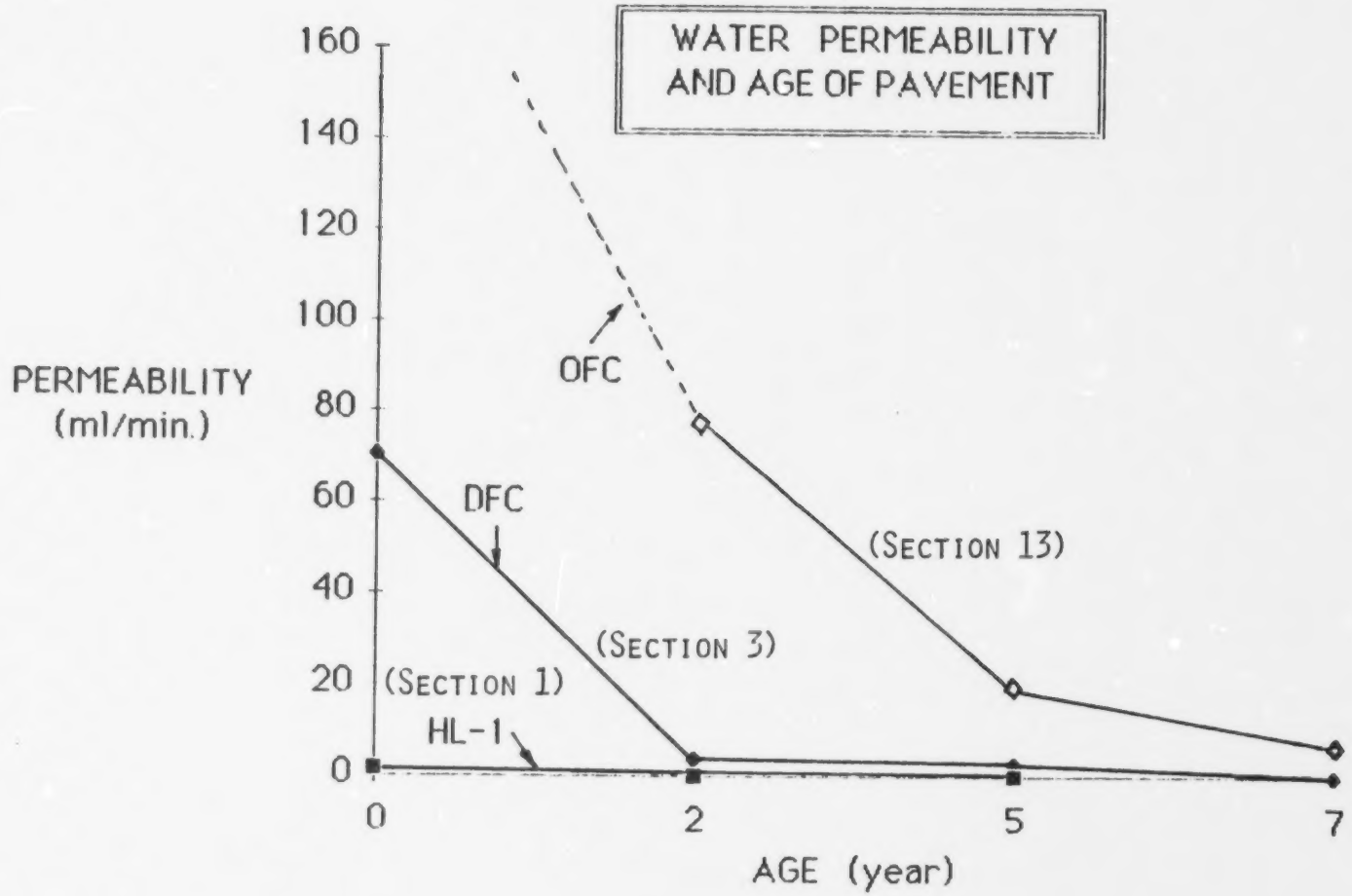


Fig. 34: Relationship Between Coarse Aggregate Content (% C.A.) and Parameter 'D' by Photointerpretation Method for the Driving, Centre and Passing Lanes

Fig. 35


WATER PERMEABILITY
AND AGE OF PAVEMENT
PERMEABILITY
(ml/min.)
160
140
120
100
80
60
40
20
0
OFC
DFC
(SECTION 13)
(SECTION 3)
(SECTION 1)
HL-1
0
2
5
7
AGE (year)

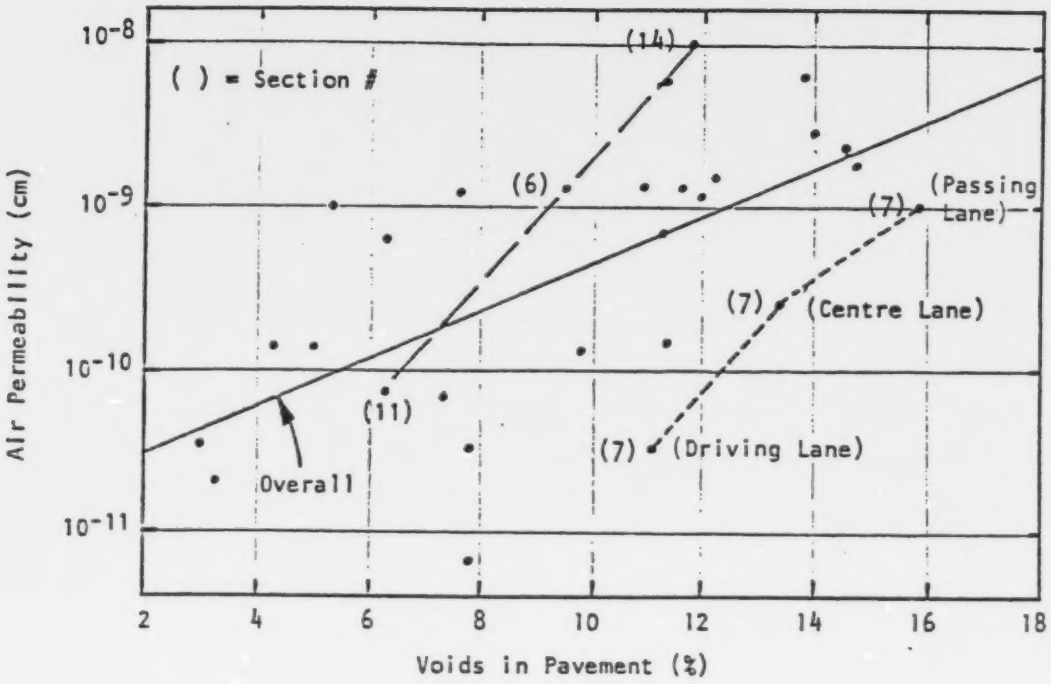


Fig. 36: Influence of Air Void Content (%) on the Air Permeability of Asphalt Pavement Materials

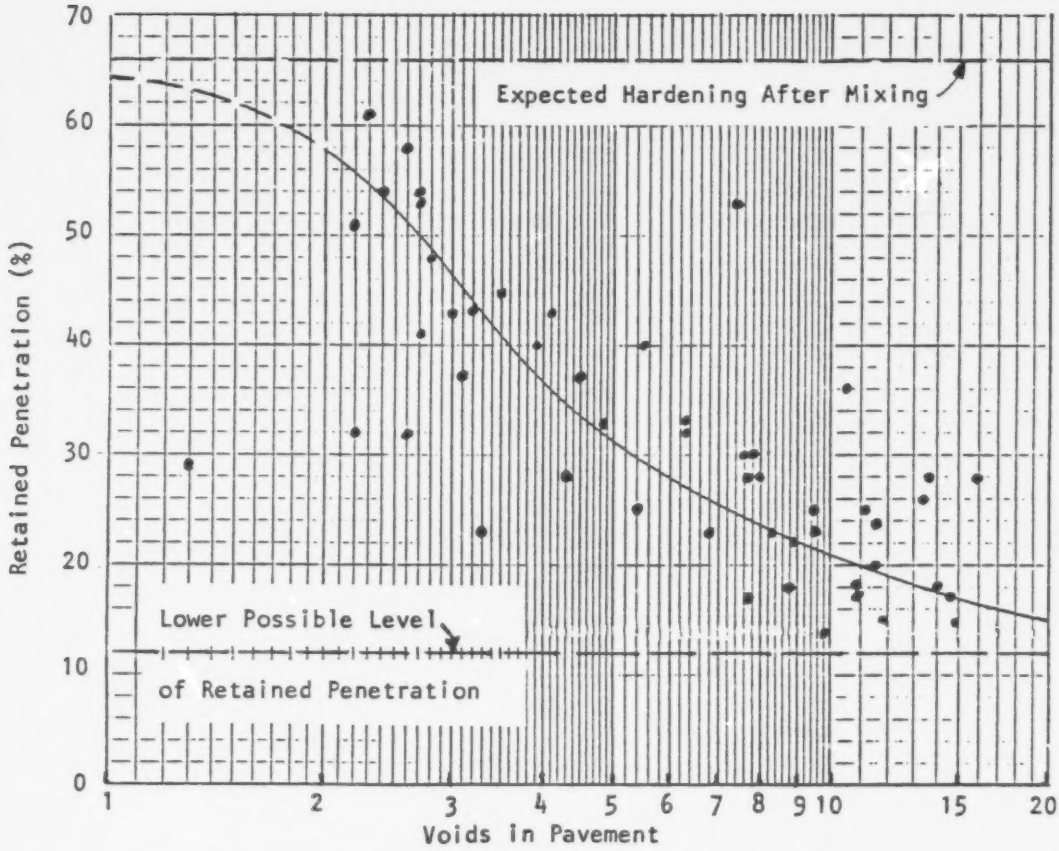


Fig. 37: Influence of Air Void Content (%) on the Retained Penetration (%, 11 yr/Original Value x 100) of Recovered Asphalt Cement From the Test Sections After 11 Years of Service

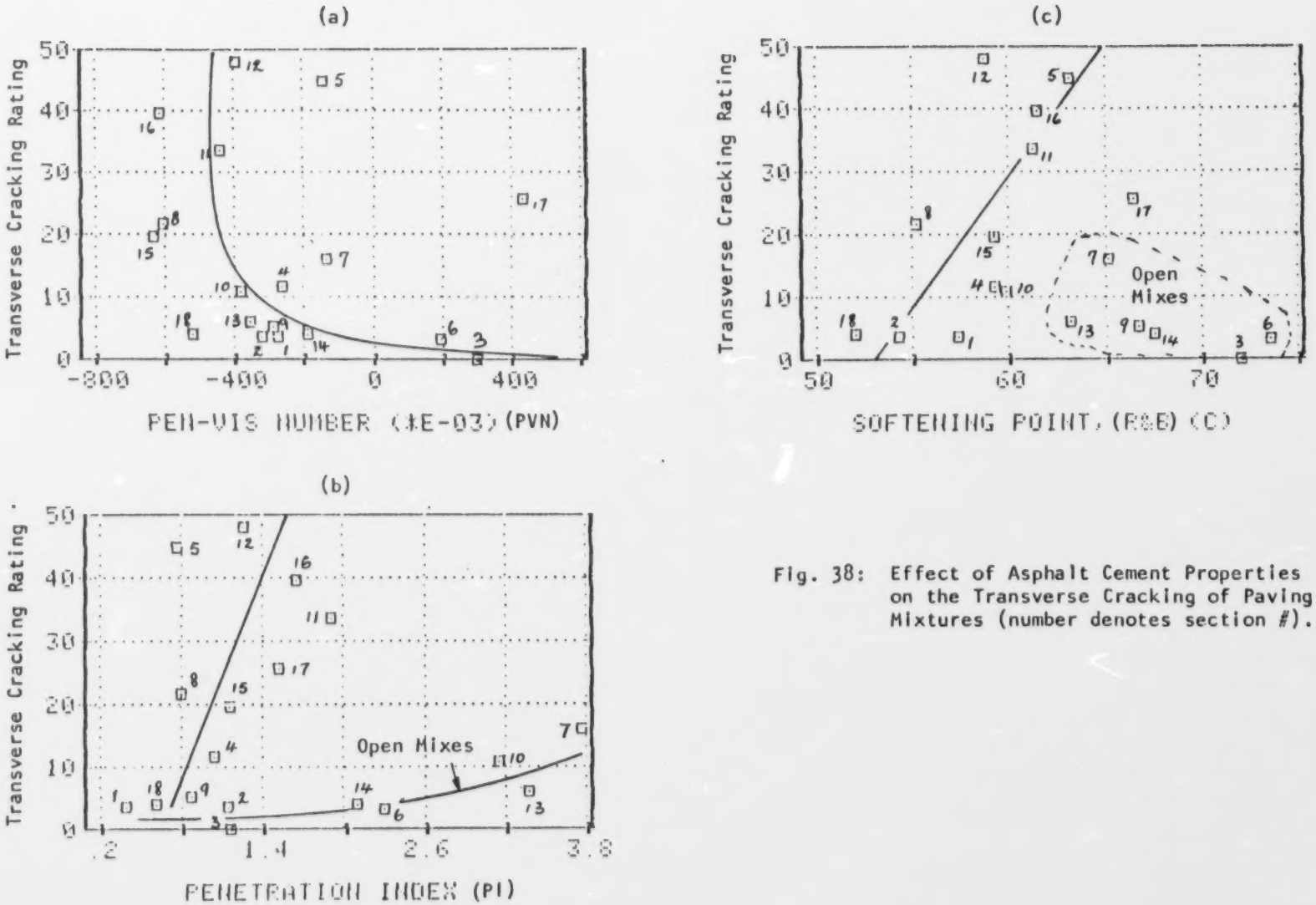
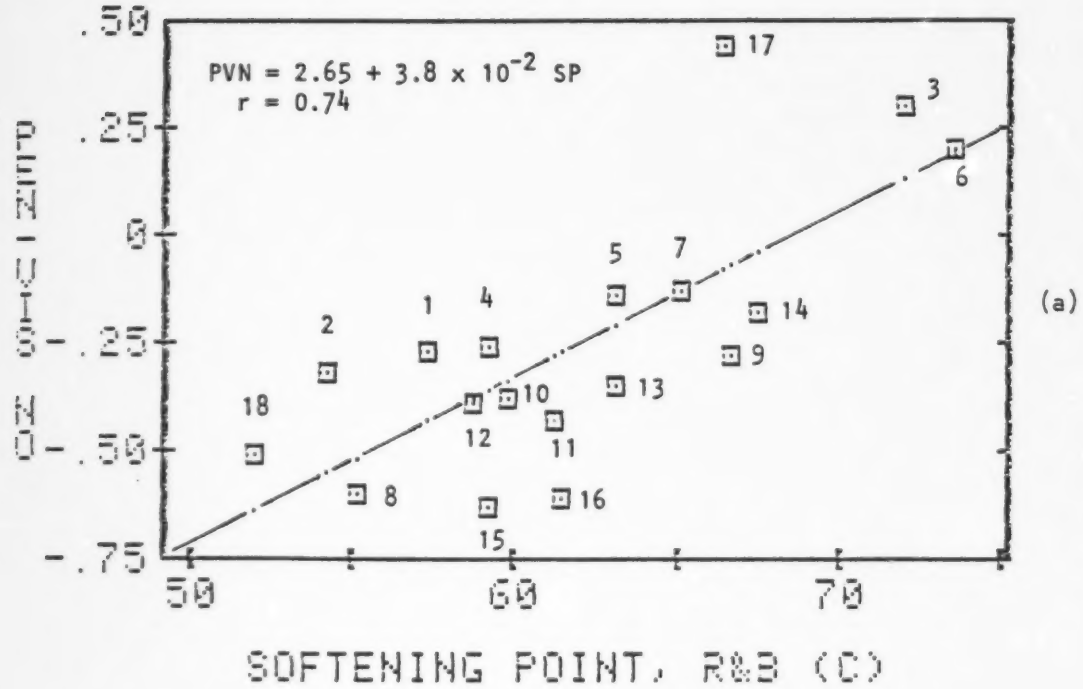


Fig. 38: Effect of Asphalt Cement Properties on the Transverse Cracking of Paving Mixtures (number denotes section #).



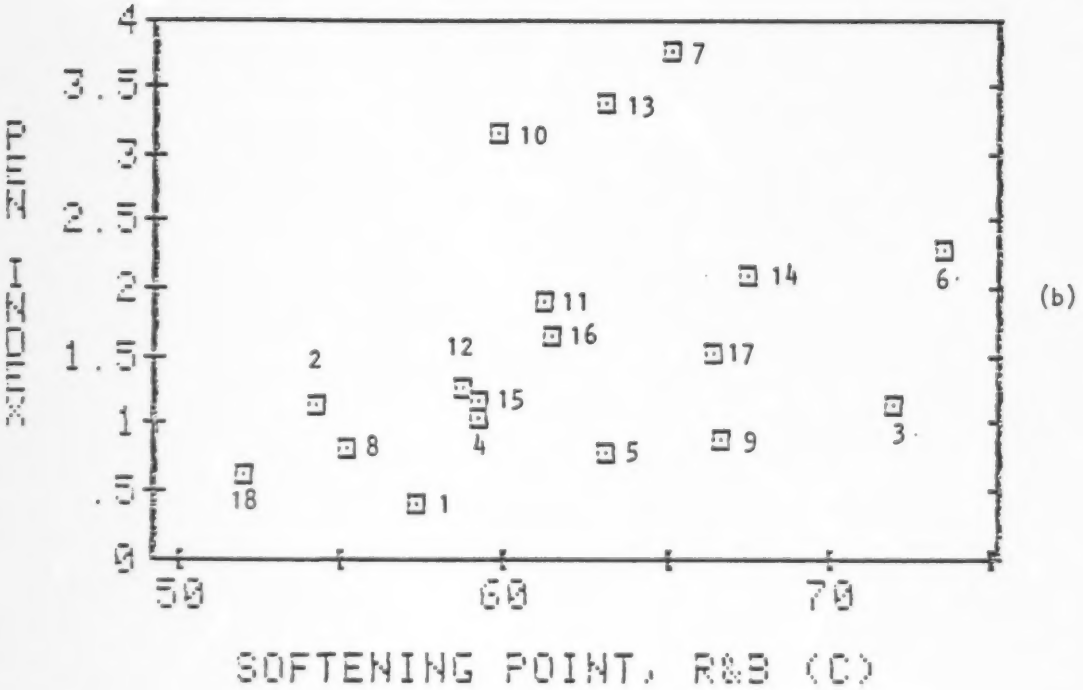


Fig. 39: Relationship Between Softening Point (R&B, °C) and Pen-Visc Number (PVN) and Penetration Index (PI)

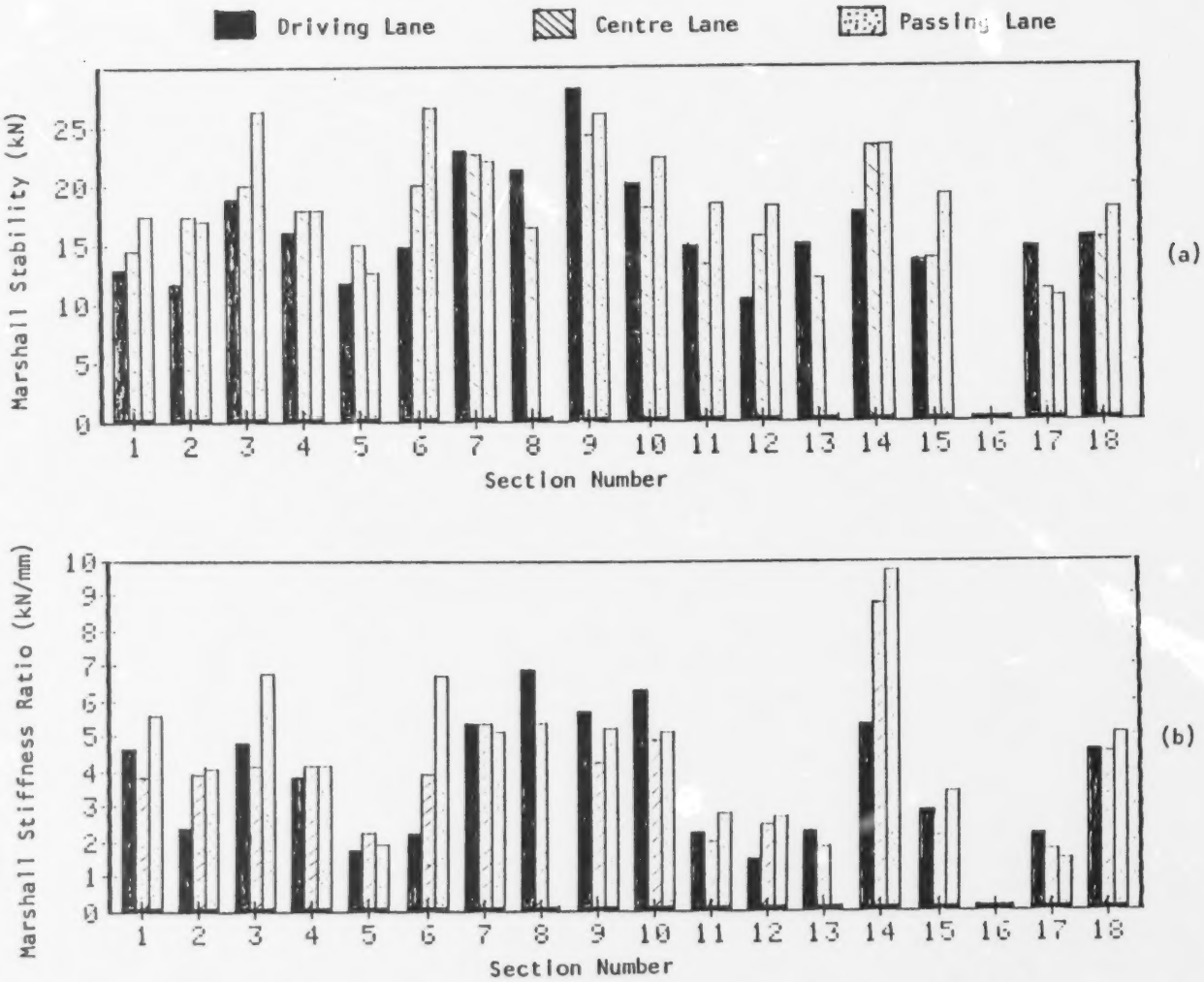

Driving Lane
Centre Lane
Passing Lane
Marshall Stability (kN)
0
5
10
15
20
25
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
Section Number
(a)
Marshall Stiffness Ratio (kN/mm)
0 1 2 3 4 5 6 7 8 9 10
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
Section Number
(b)

Fig. 40: Marshall Stability and Stiffness Ratio of Experimental Mixes (on Recompacted Samples at 11th Year)

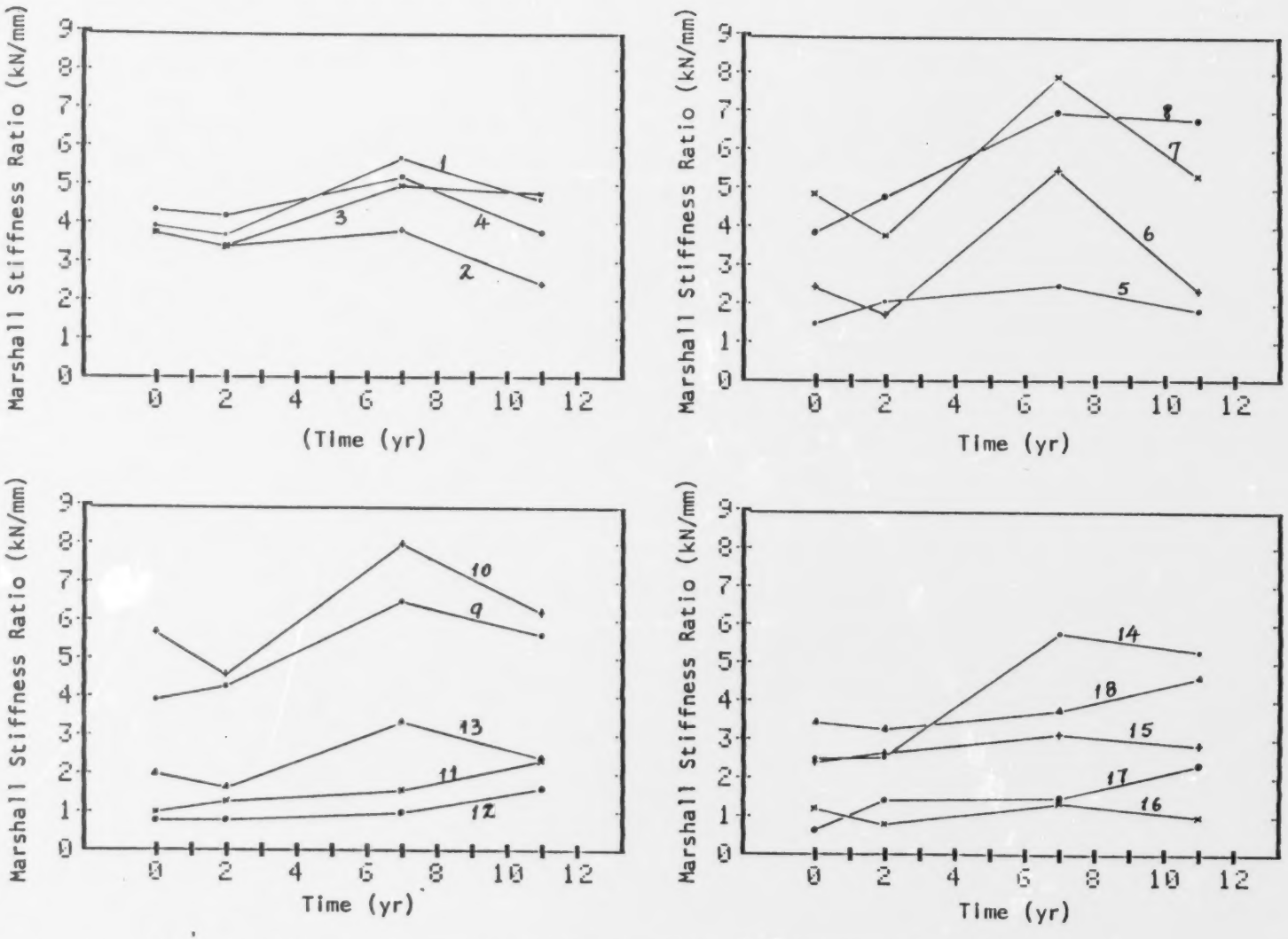


Fig. 41: Change of Marshall Stiffness Ratio of Recompacted Samples (from the Driving Lane) With Time (year) (number denotes section #).

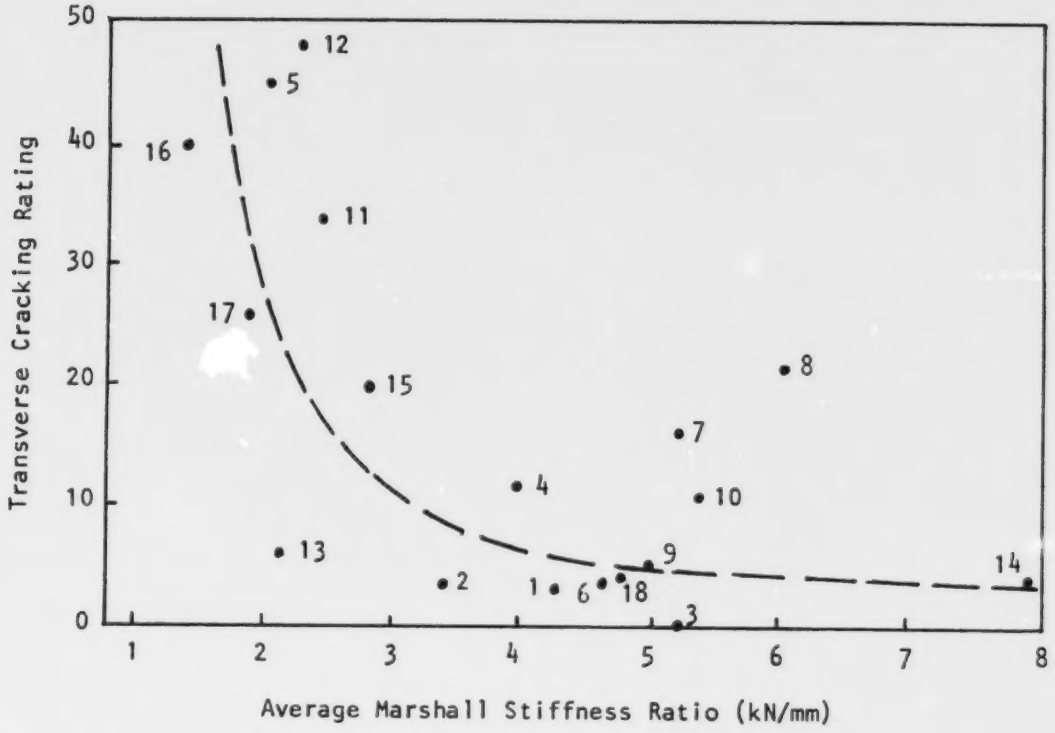


Fig. 42: Relationship Between Transverse Cracking Rating and Marshall Stiffness Ratio of Recompacted Samples.

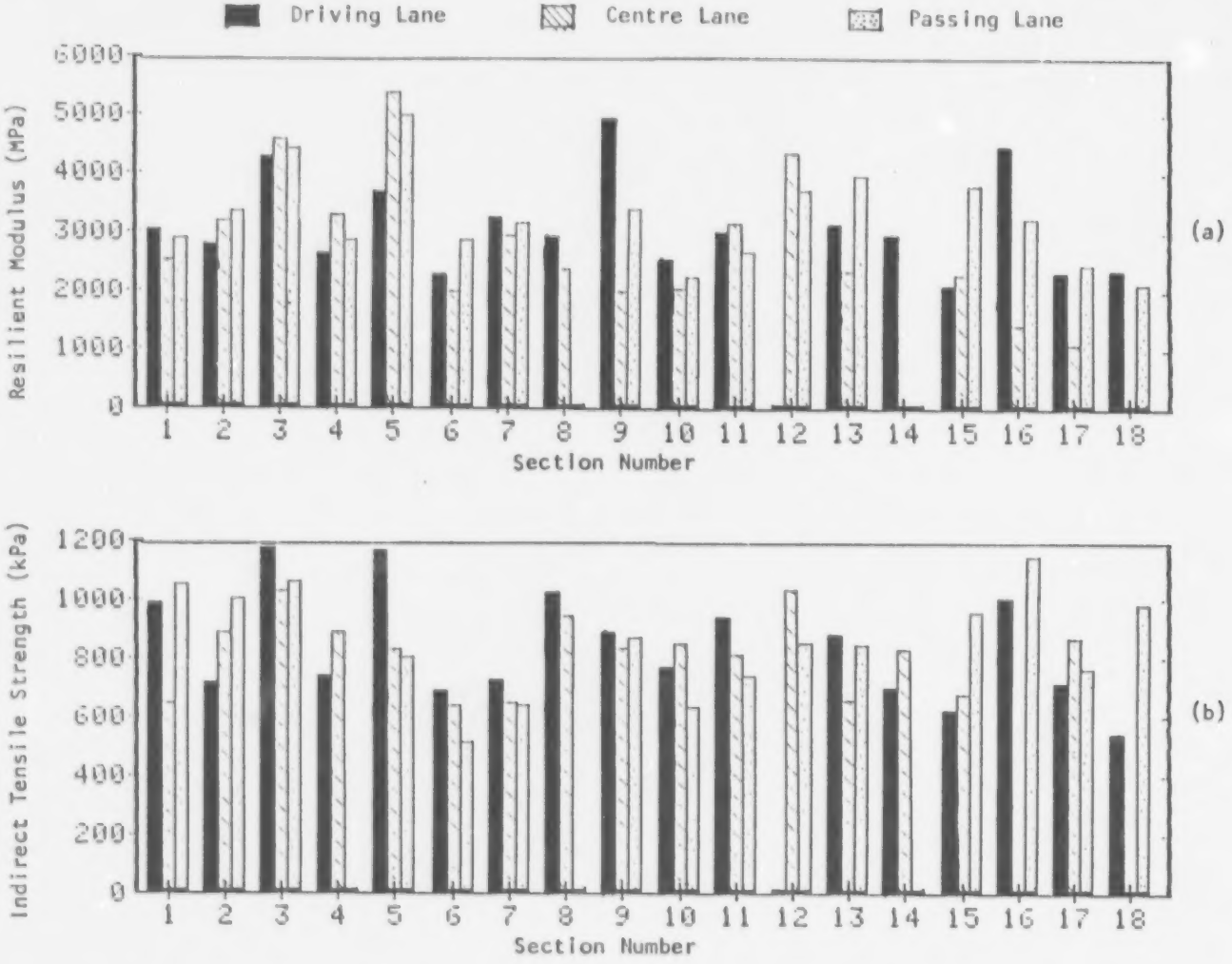


Fig. 43: Resilient Modulus and Indirect Tensile Strength of Different Test Sections (on 4"dia. Cores at 23°C).

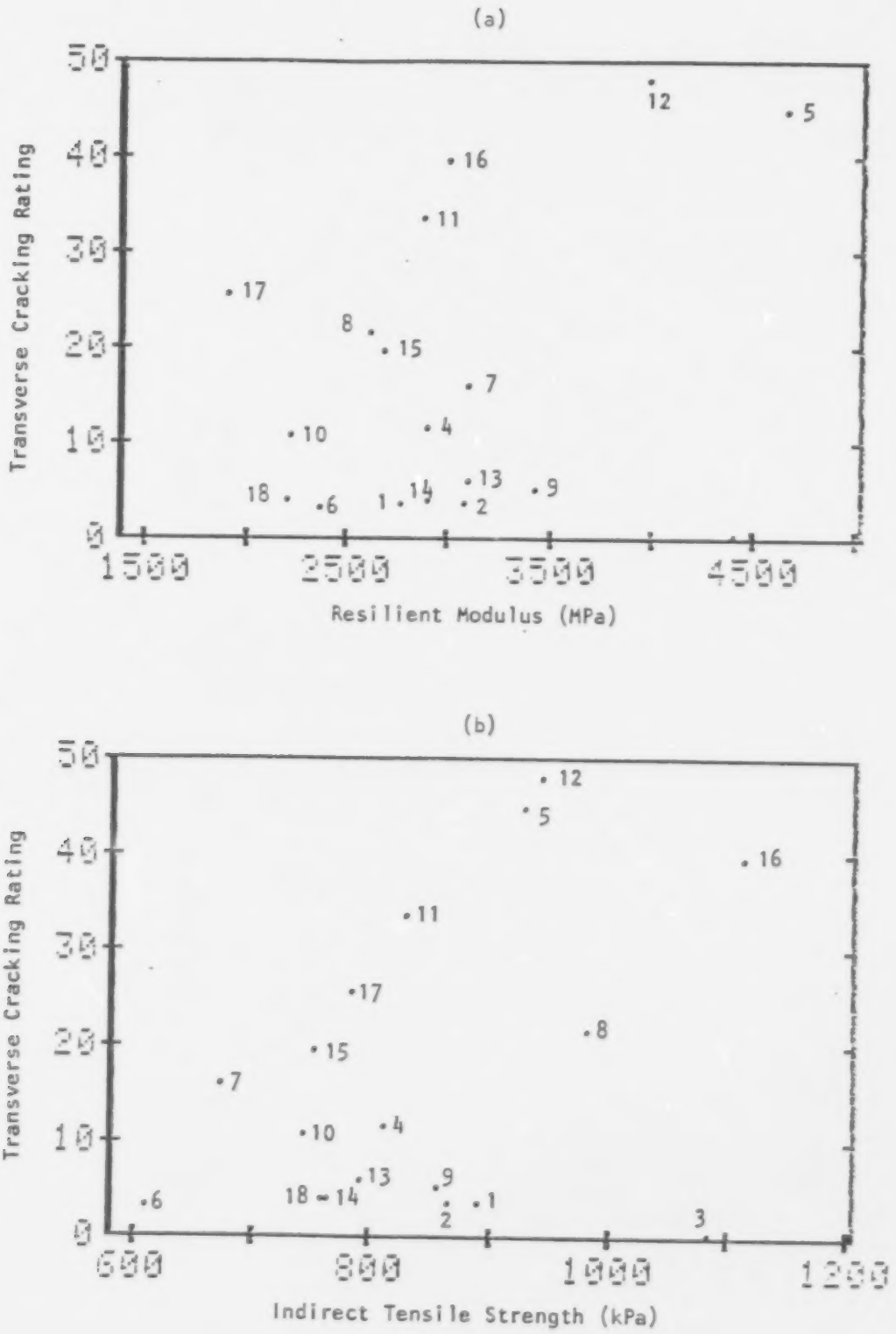


Fig. 44: Mix Resilient Modulus and Indirect Tensile Strength vs Transverse Cracking Rating (at 11 Years, Number = Section #).

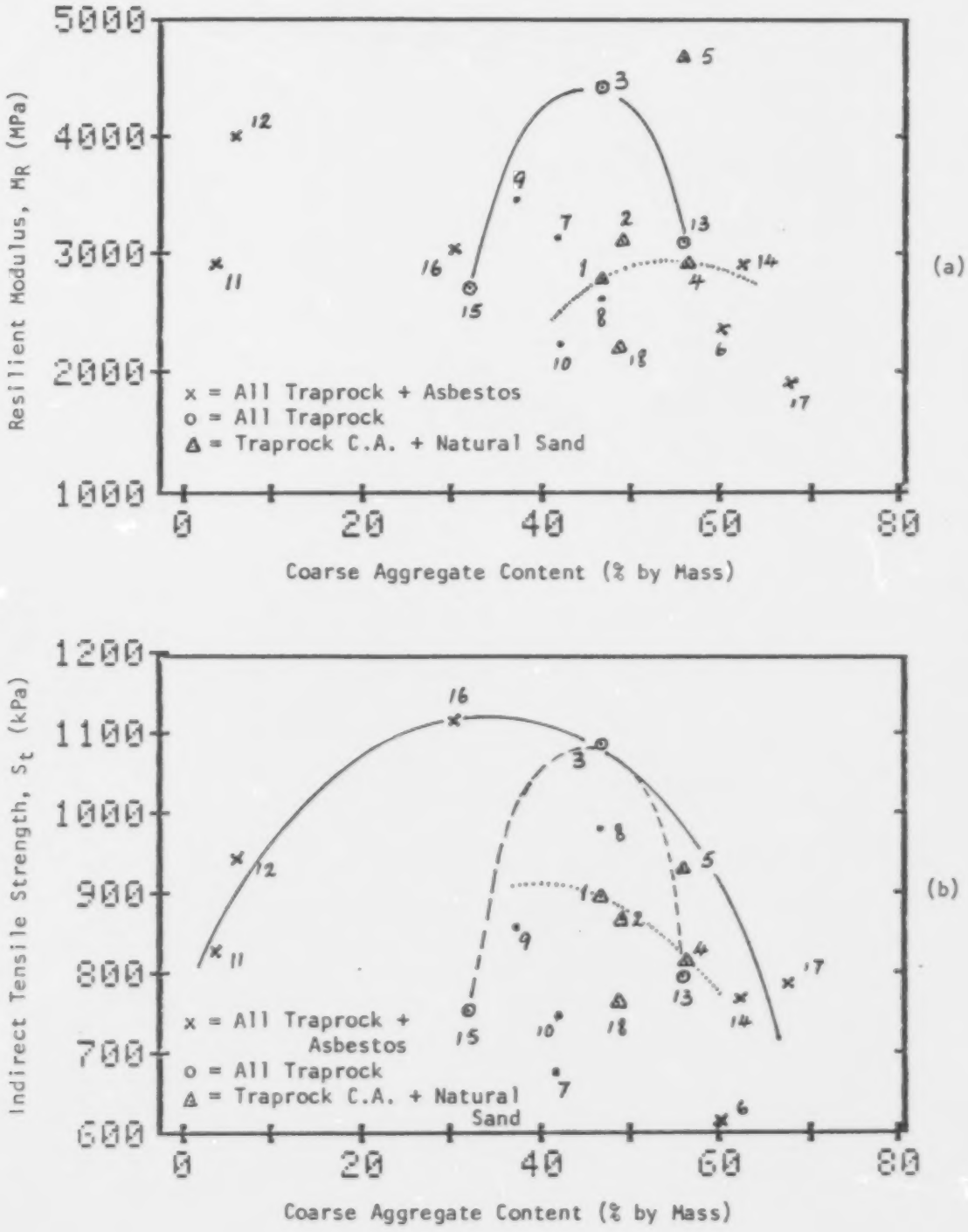


Fig. 45: Influence of Coarse Aggregate Content on the Resilient Modulus and Indirect Tensile Strength of Asphalt Mixtures

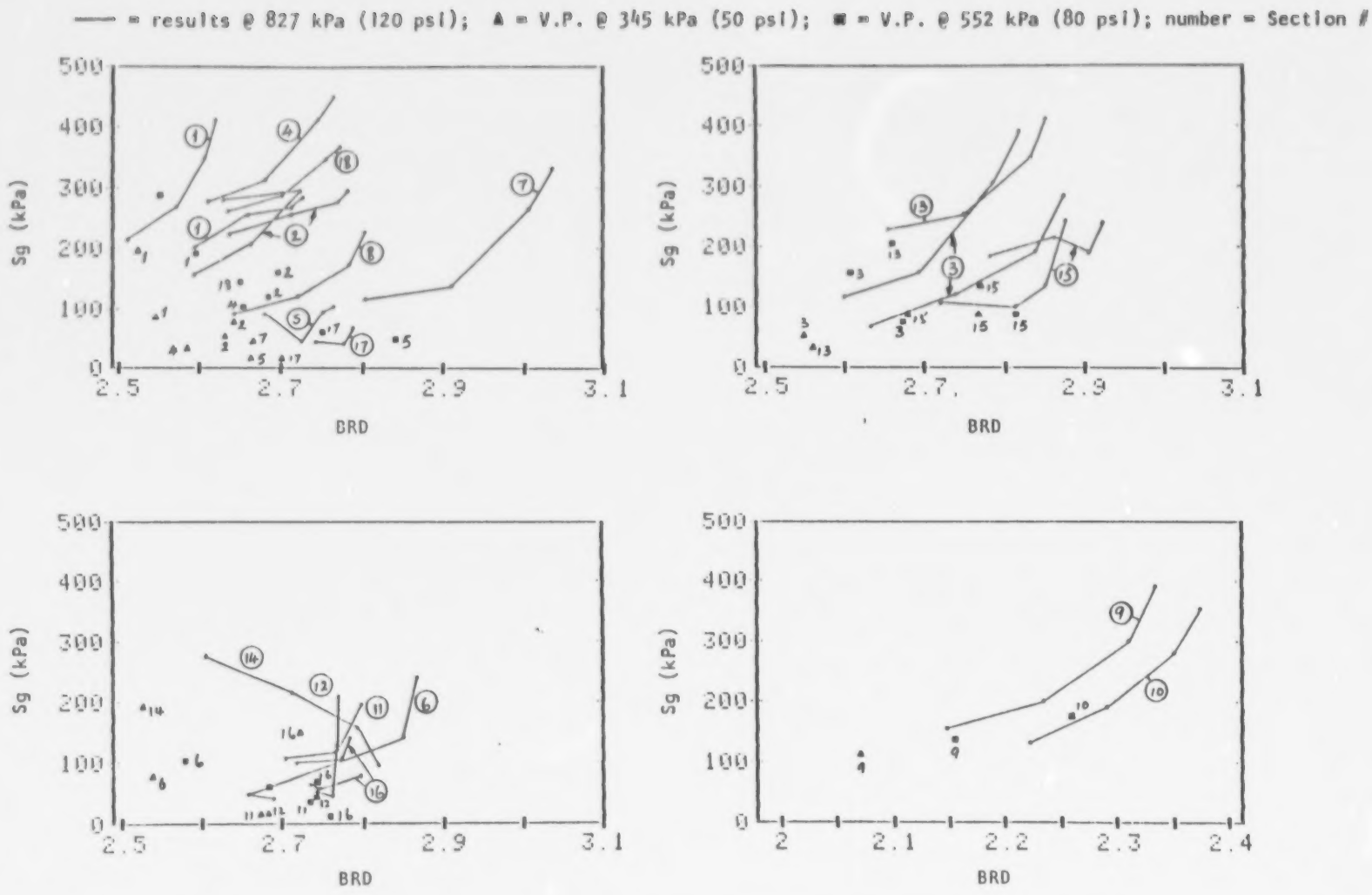


Fig. 46: Change in Gyratory Shear Strength (Sg, kPa) with Bulk Relative Density (BRD)

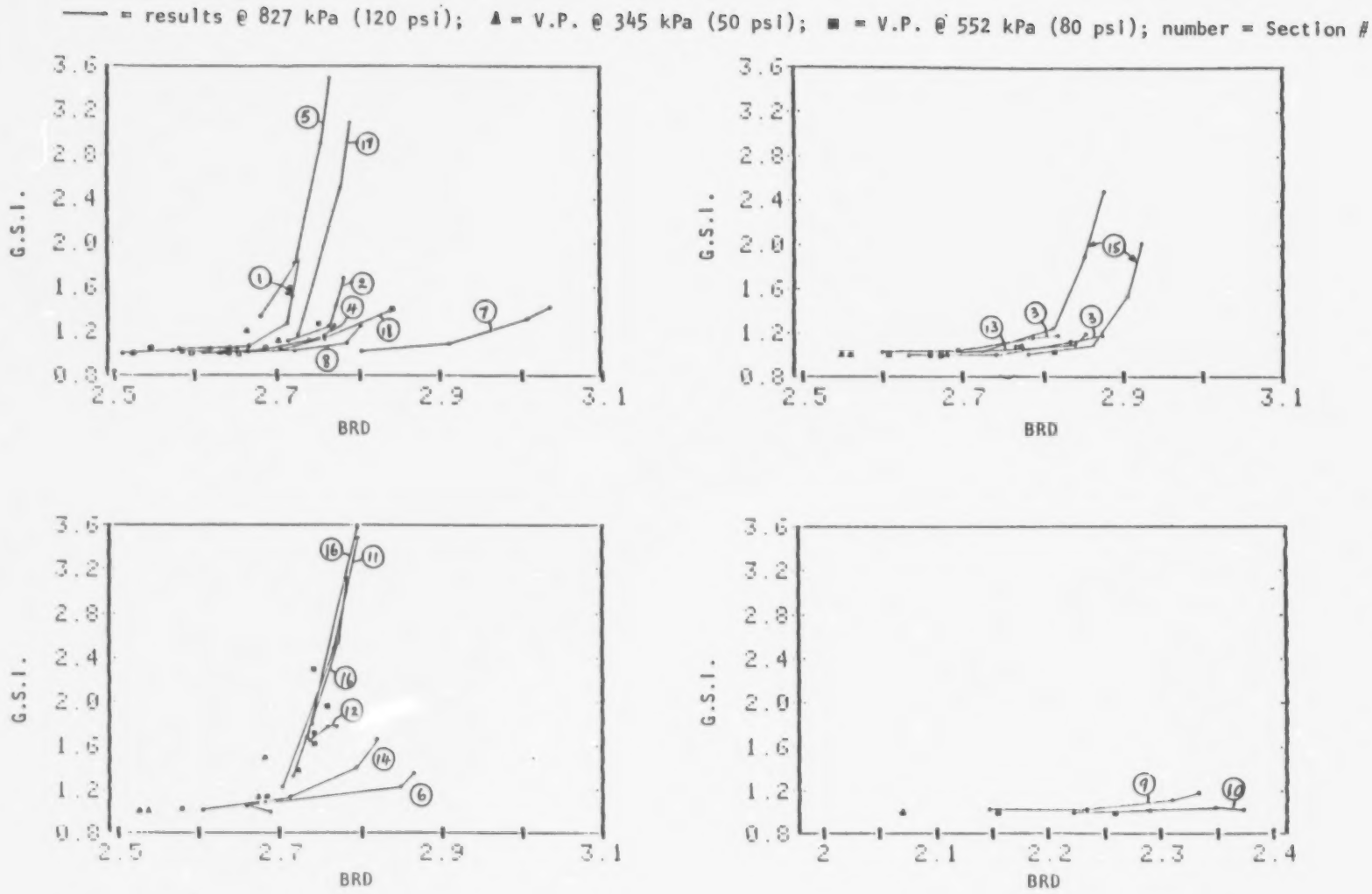


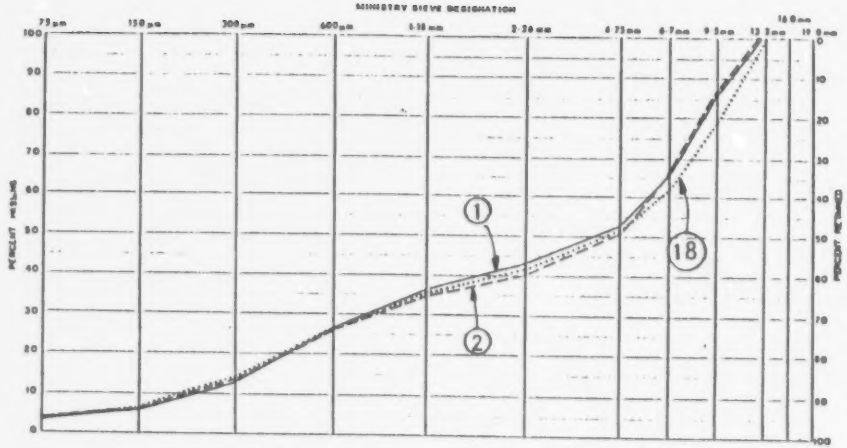
Fig. 47: Change in Gyratory Stability Index (G.S.I.) with Bulk Relative Density (BRD)

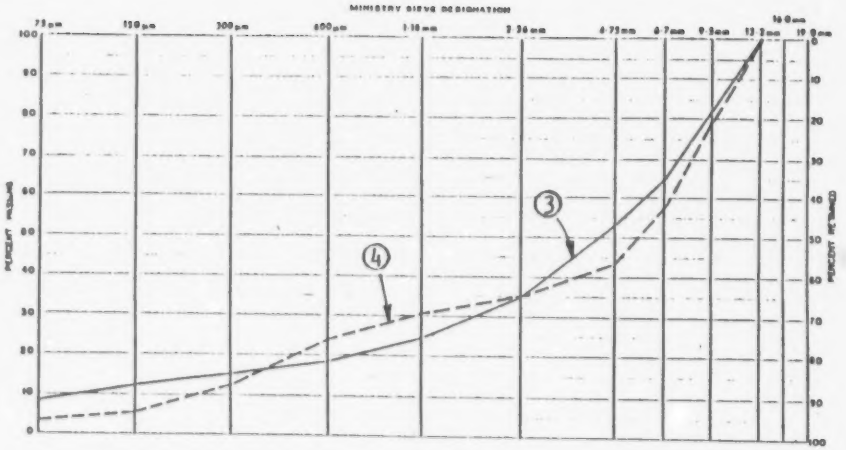
APPENDIX 1

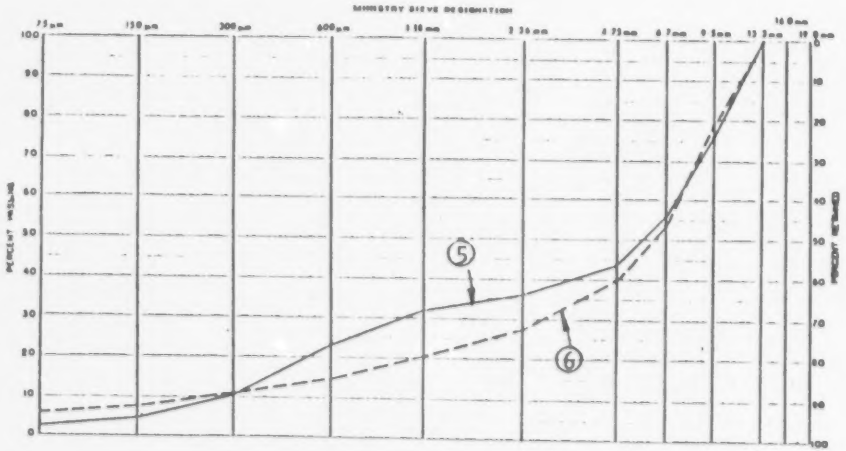
Aggregation Gradation of Cores

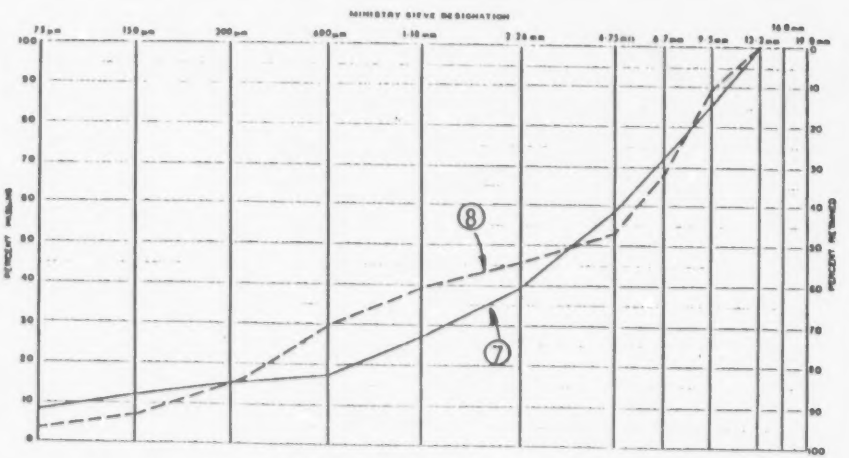
Taken in 1985

Semi-Log Gradation Chart





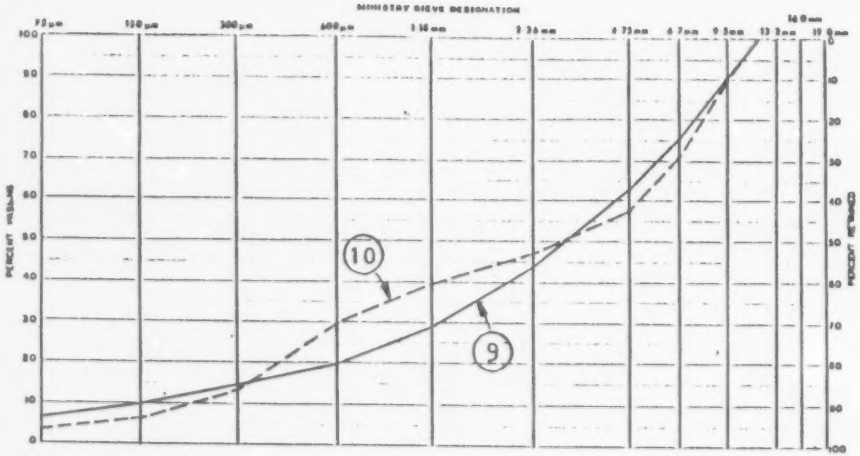


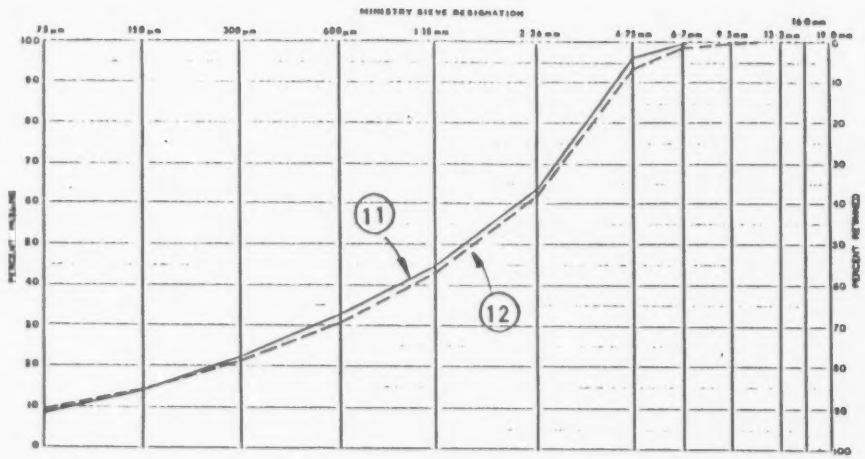


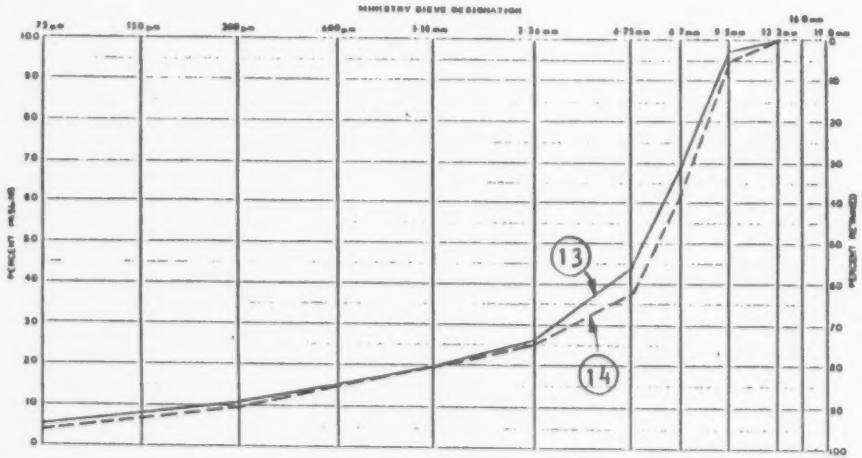
* Figure in O = Section Number

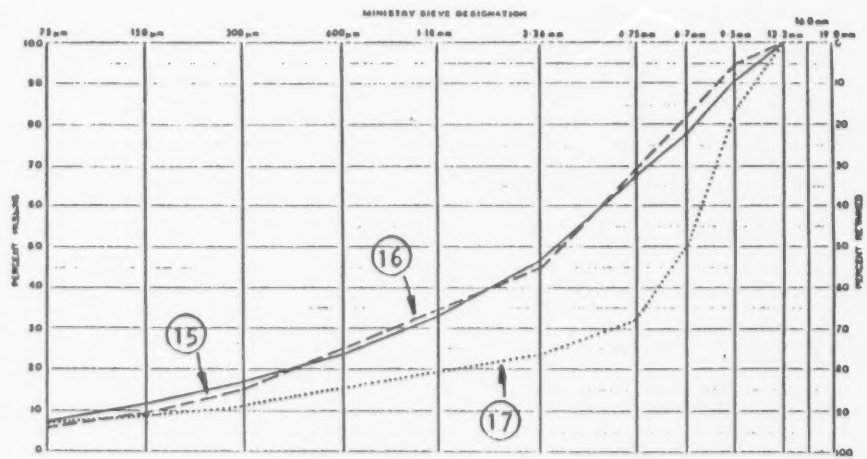
* Plotted Value = average of cores from the three lanes

Semi-Log Gradation Chart









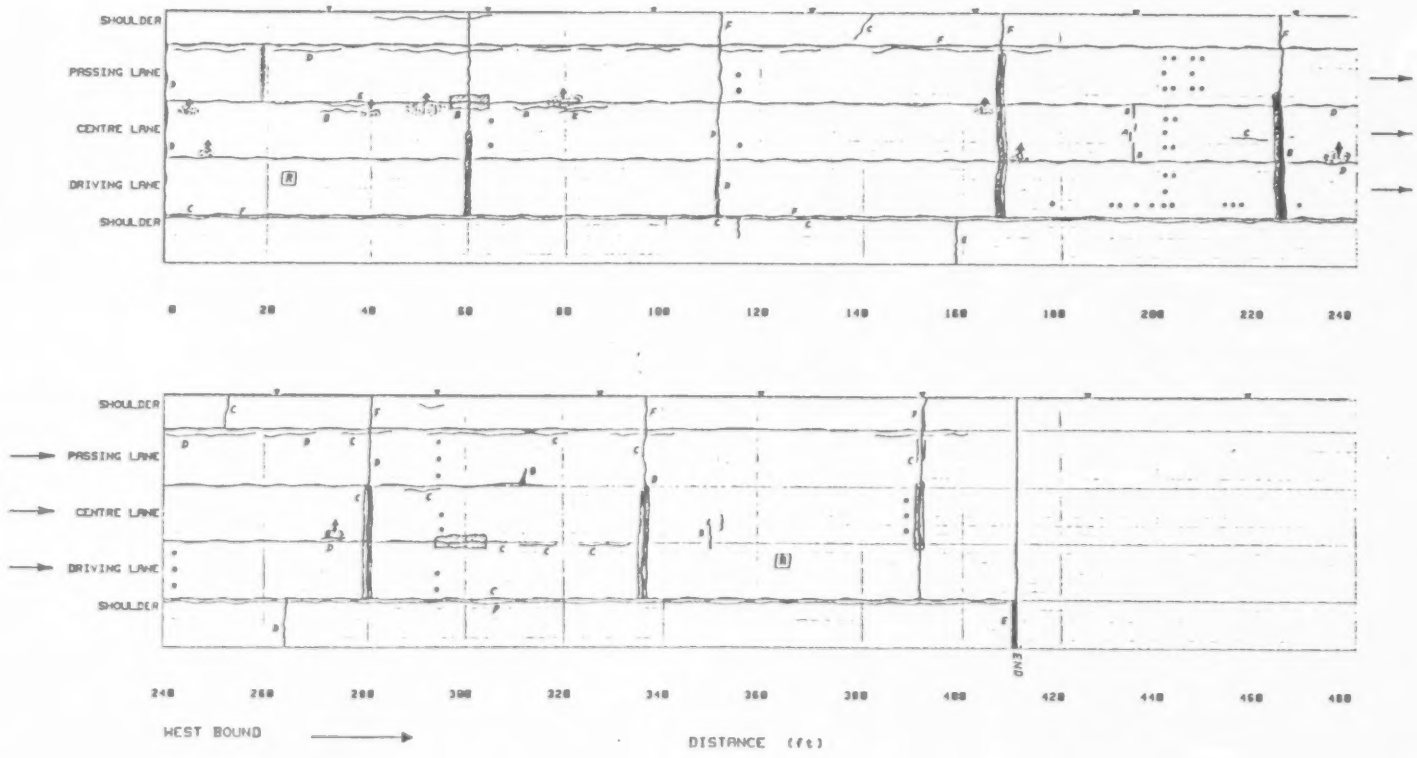
* Figure In O = Section Number

* Plotted Value = average of cores from the three lanes

APPENDIX 2

Condition Survey Map

of the 18 Test Sections

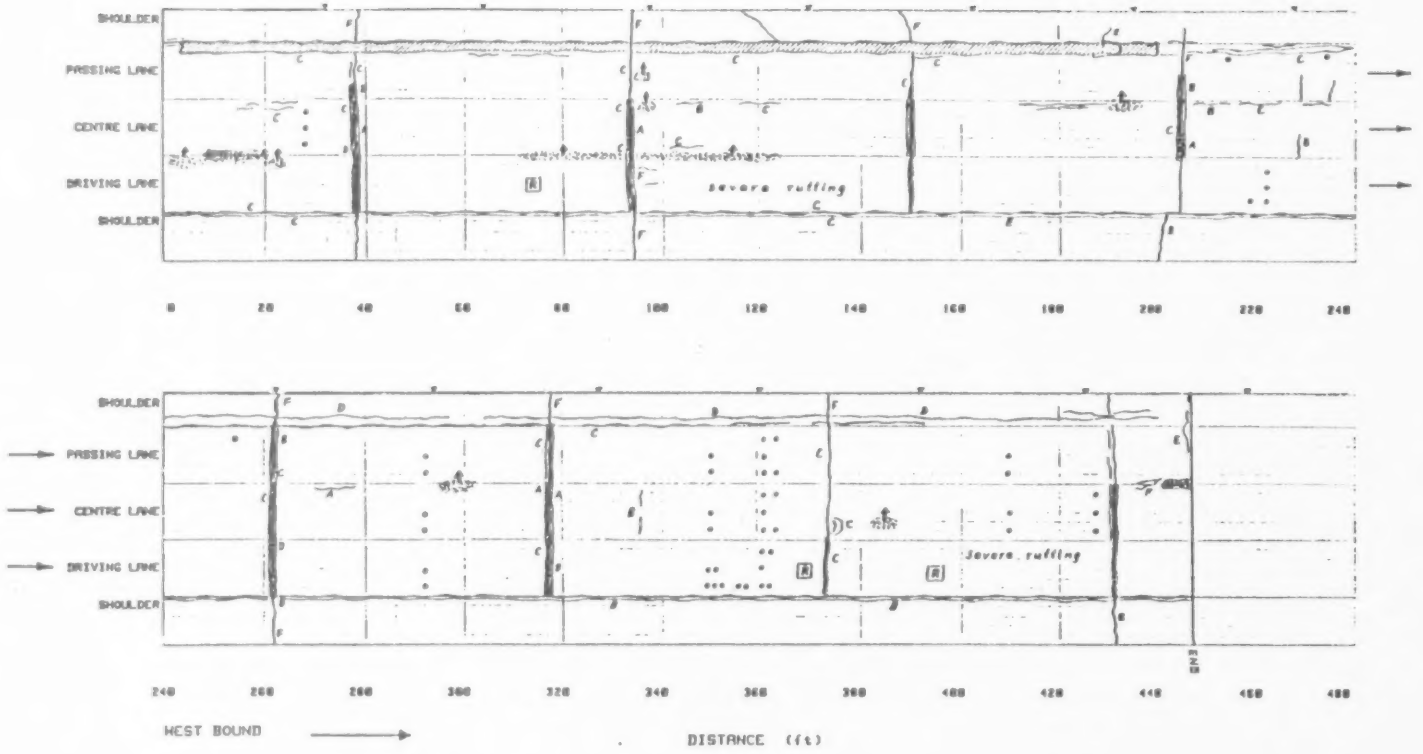


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 1

KEY

- CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E= 0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  I= treated
- STEPPING
- SETTLEMENT, HEAVING, ETC.
- BLOW-UP
- CONSTRUCTION JOINT
- RAVELLING
- DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
- PATCHING
- OVERLAY
- CORE TAKEN
- RUT DEPTH MEASURED [A]

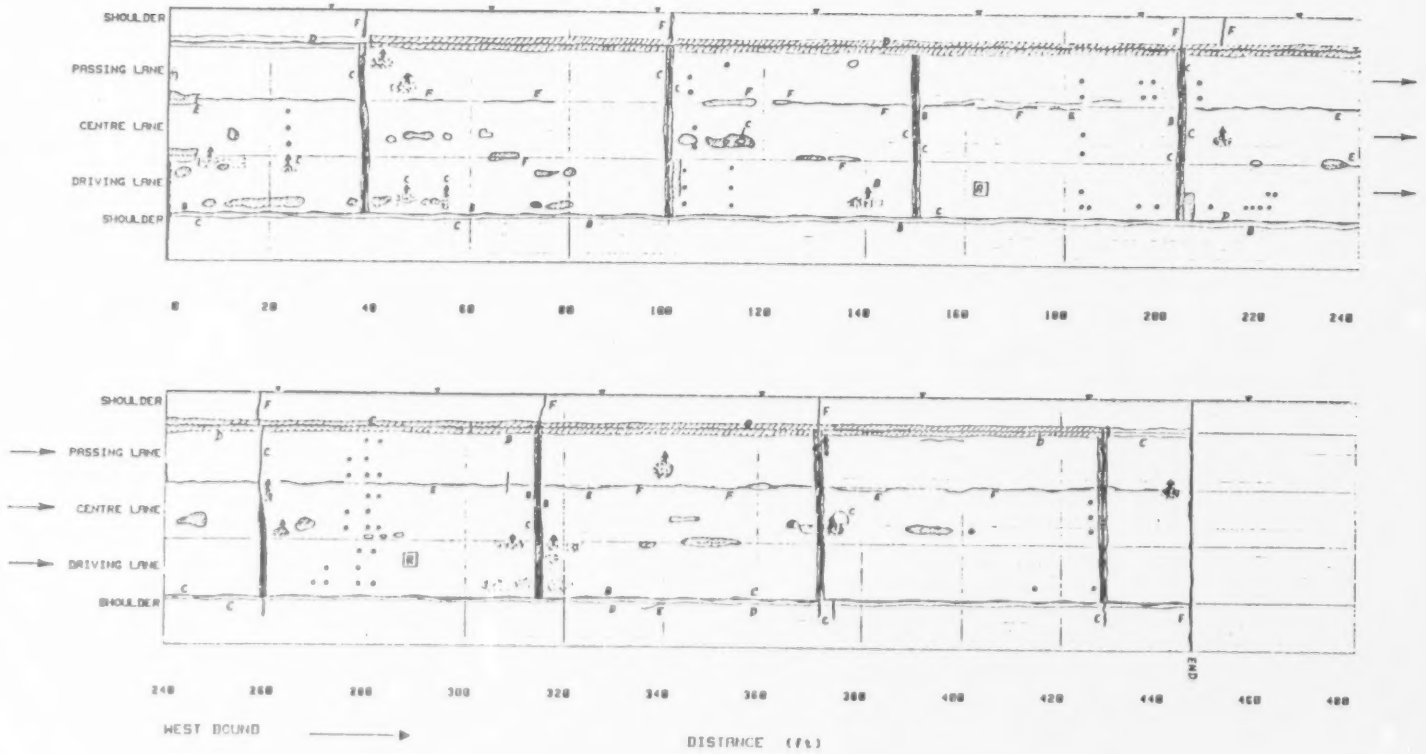


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 2

KEY

- CRACKING
  Code: A= hairline; B= < 0.2" (< 5mm); C= 0.2-0.4" (5-10mm); D= 0.4-0.6" (10-15mm); E=0.6-0.8" (15-20mm); F= > 0.8" (> 20mm); H= healed over; T= treated
- STEPPING
- SETTLEMENT, HEAVING, ETC.
- BLOW-UP
- CONSTRUCTION JOINT
- RAVELLING
- DELAMINATION
  Code: A= slight; B= Medium; C= severe
- PATCHING
- OVERLAY
- CORE TAKEN
- RUT DEPTH MEASURED [R]

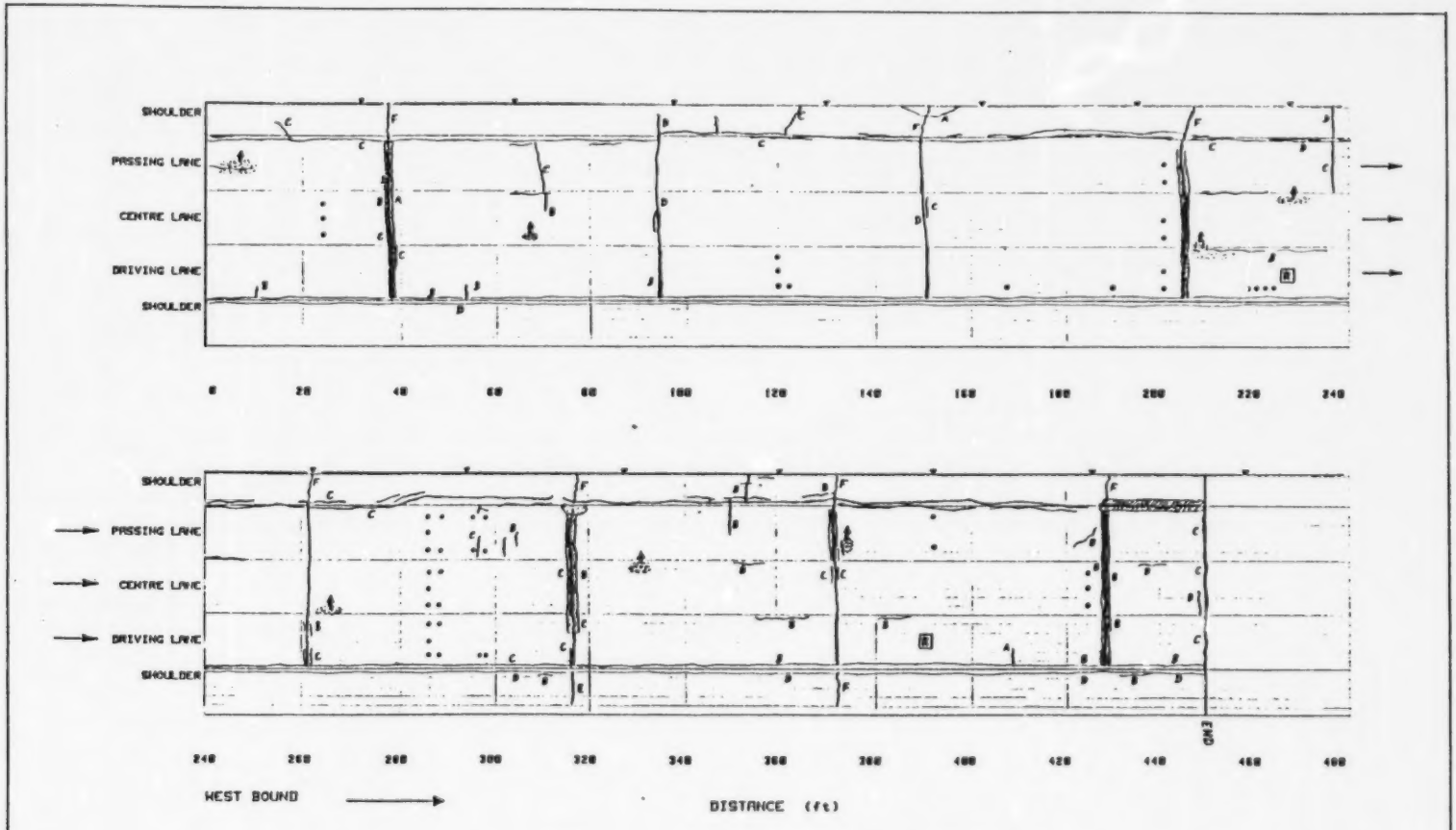


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 3

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  I= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED

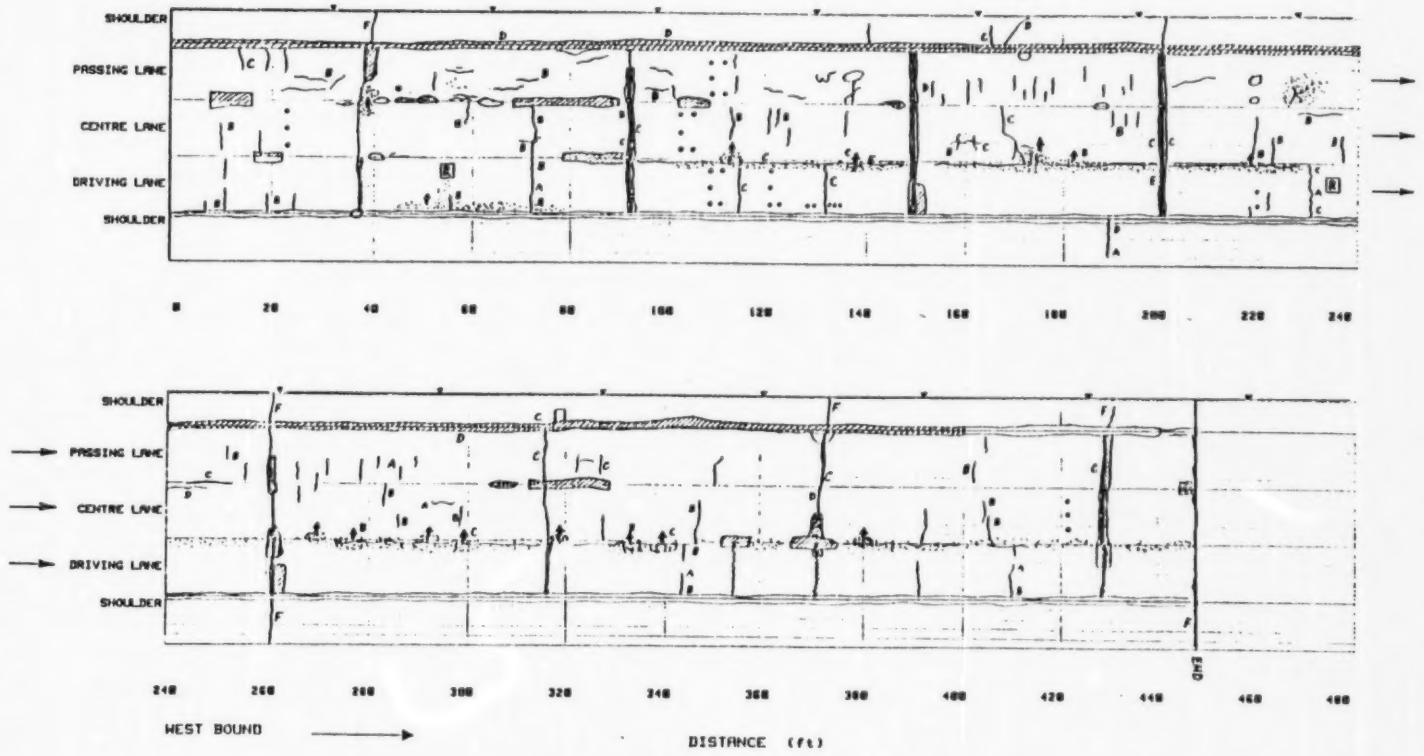


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 4

KEY

* CRACKING
  Code: A= hairline; B= < 0.2" (< 5mm); C= 0.2-0.4" (5-10mm); D= 0.4-0.6" (10-15mm); E=0.6-0.8" (15-20mm); F= > 0.8" (> 20mm); W= healed over; T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight; B= Medium; C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED [R]

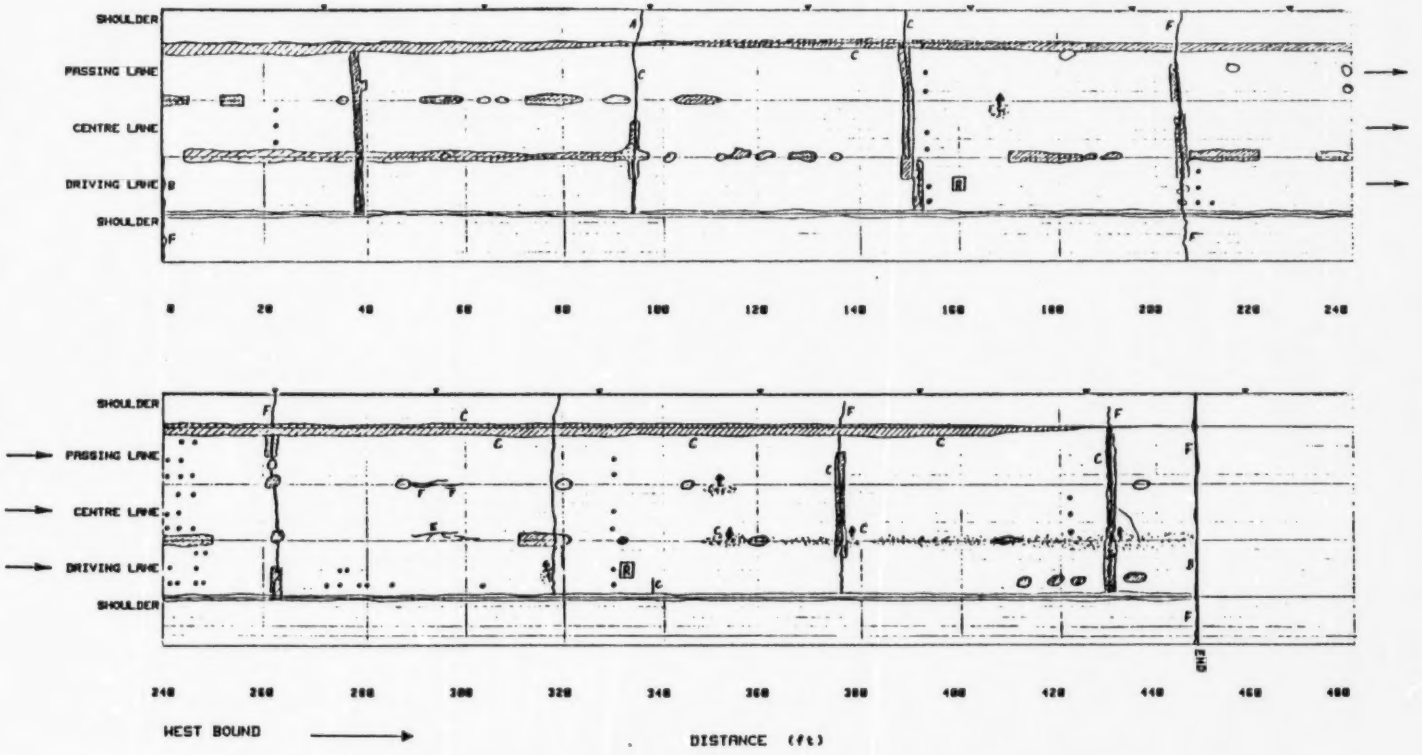


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 5

KEY

* CRACKING
  Code: A= Hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= Healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED [A]

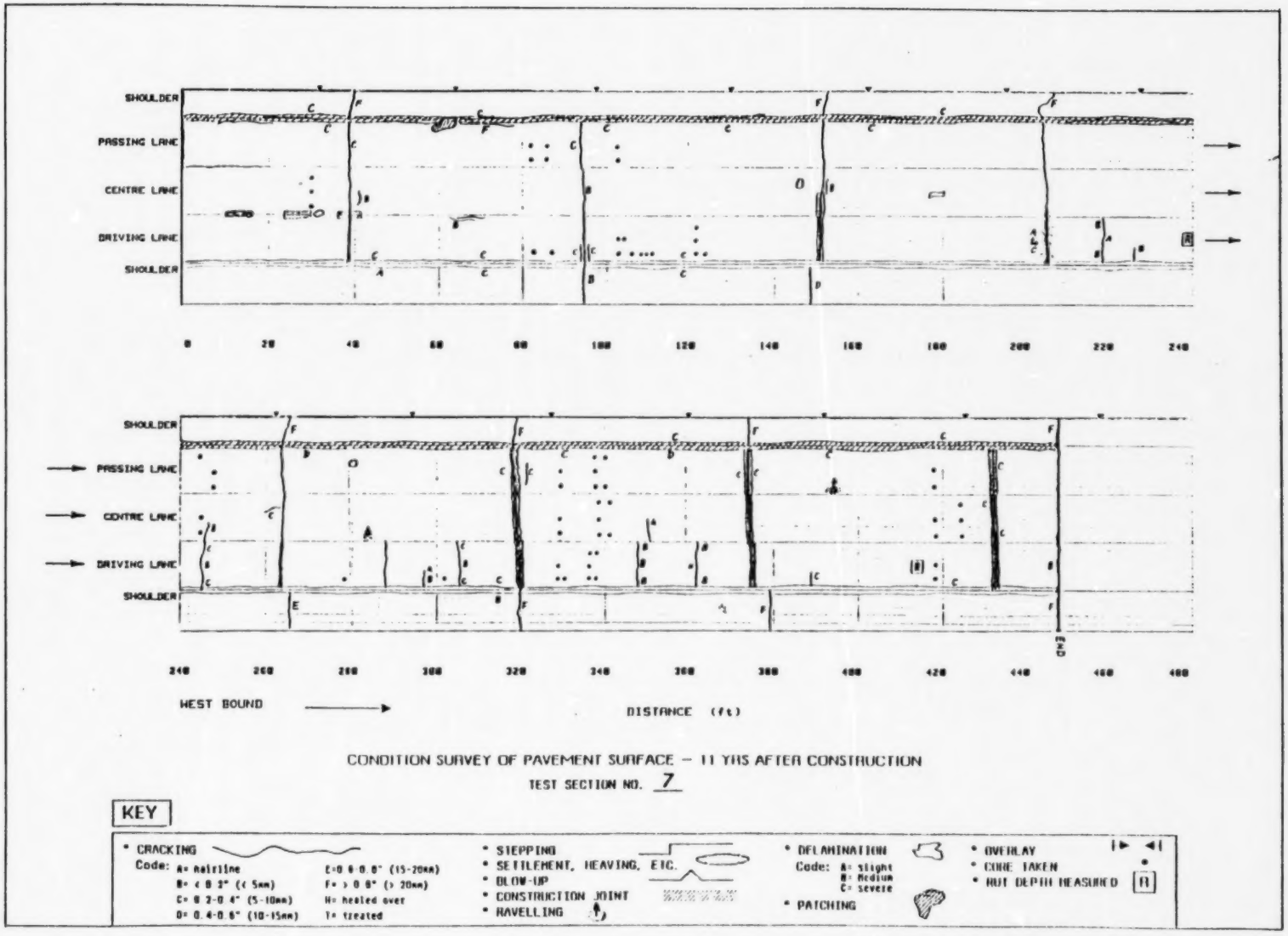


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 6

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED R

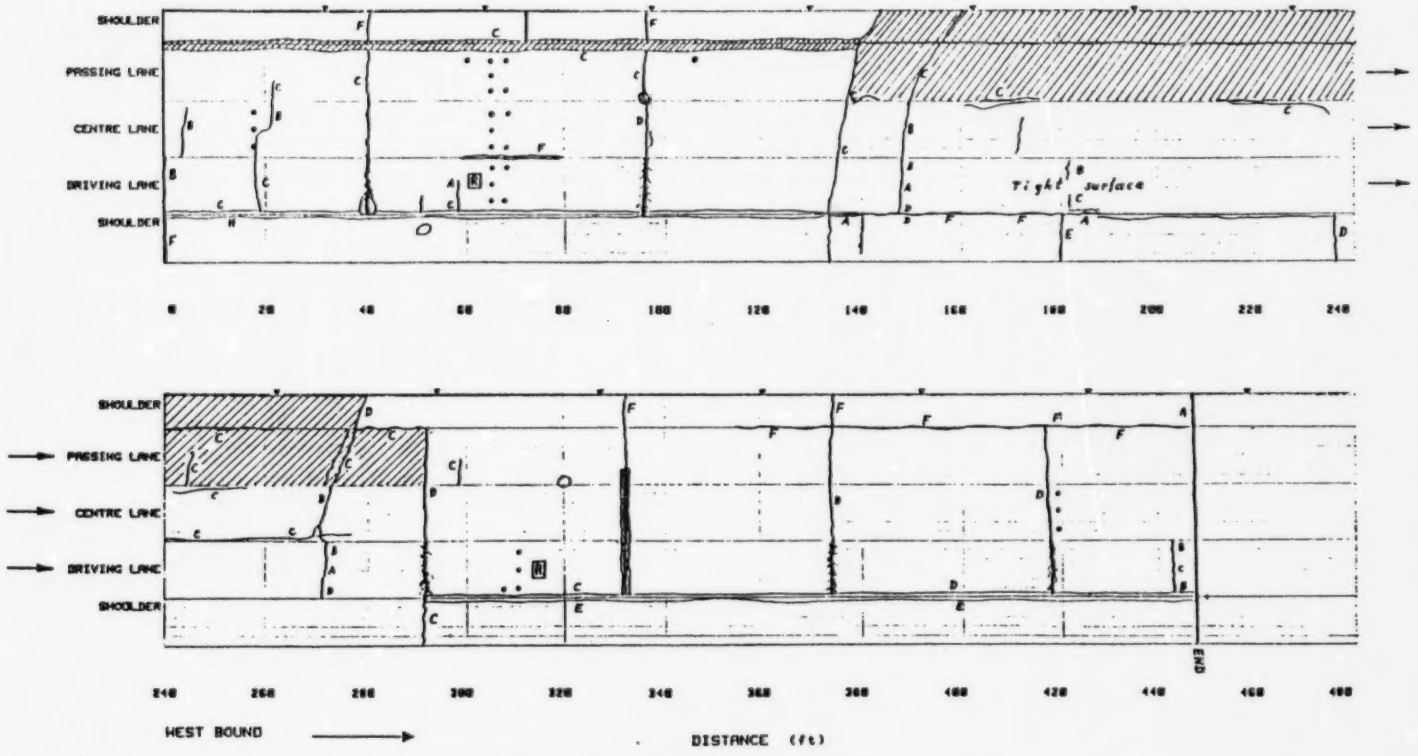


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 7

KEY

* CRACKING
  Code: A= Hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED [A]

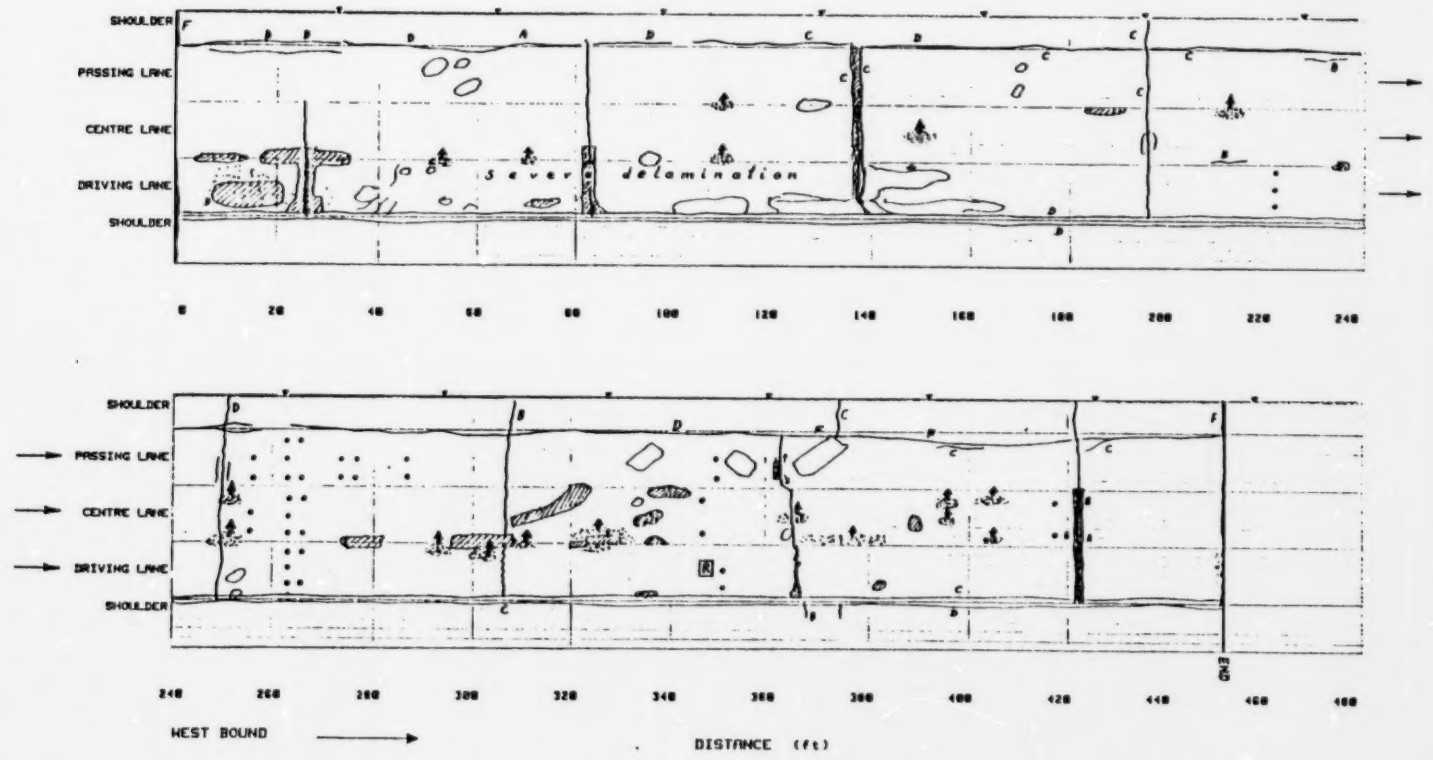


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 8

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED

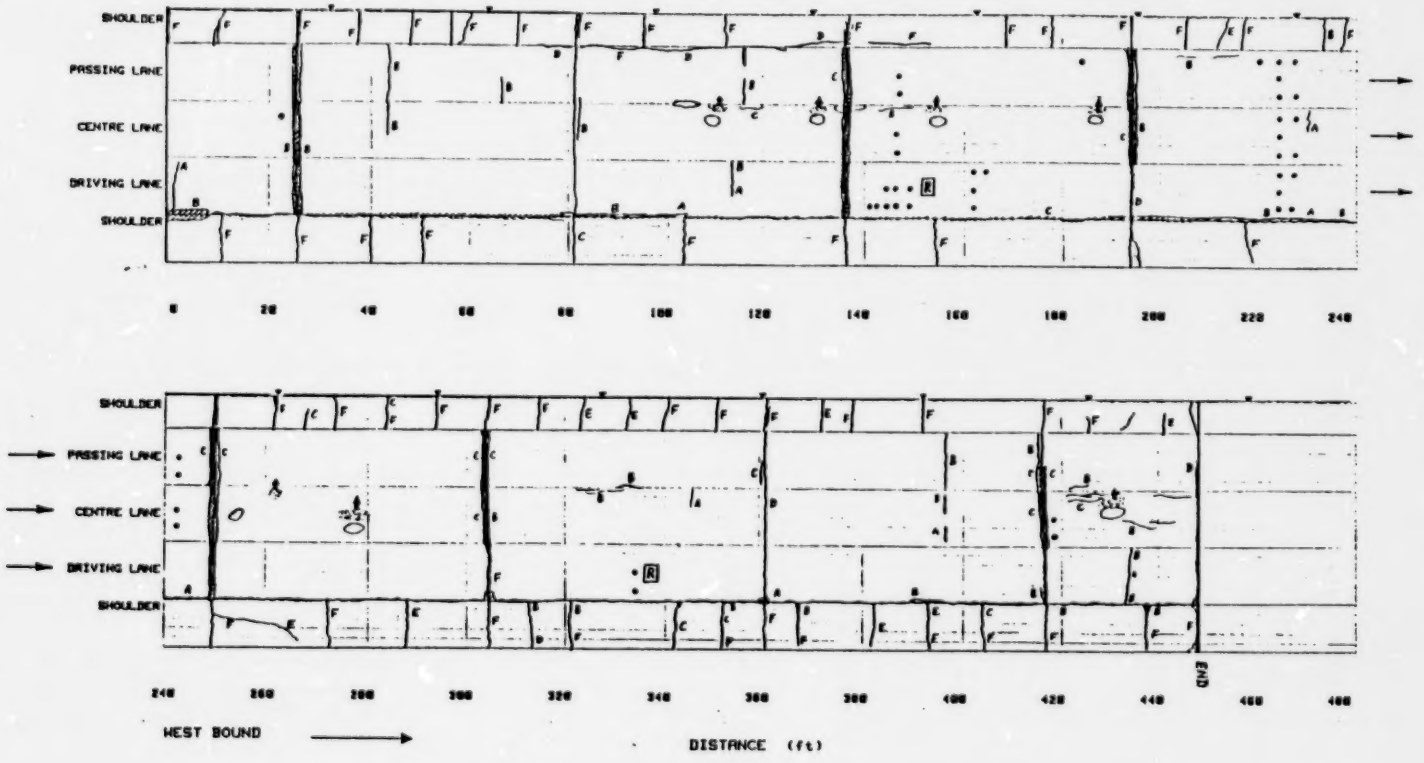


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 9

KEY

- CRACKING
  - Code: A= hairline
  - B= < 0.2" (< 5mm)
  - C= 0.2-0.4" (5-10mm)
  - D= 0.4-0.6" (10-15mm)
  - E=0.6-0.8" (15-20mm)
  - F= > 0.8" (> 20mm)
  - H= healed over
  - T= treated
- STEPPING
- SETTLEMENT, HEAVING, ETC.
- BLOW-UP
- CONSTRUCTION JOINT
- RAVELLING
- DELAMINATION
  - Code: A= slight
  - B= Medium
  - C= severe
- PATCHING
- OVERLAY
- CORE TAKEN
- RUT DEPTH MEASURED [R]

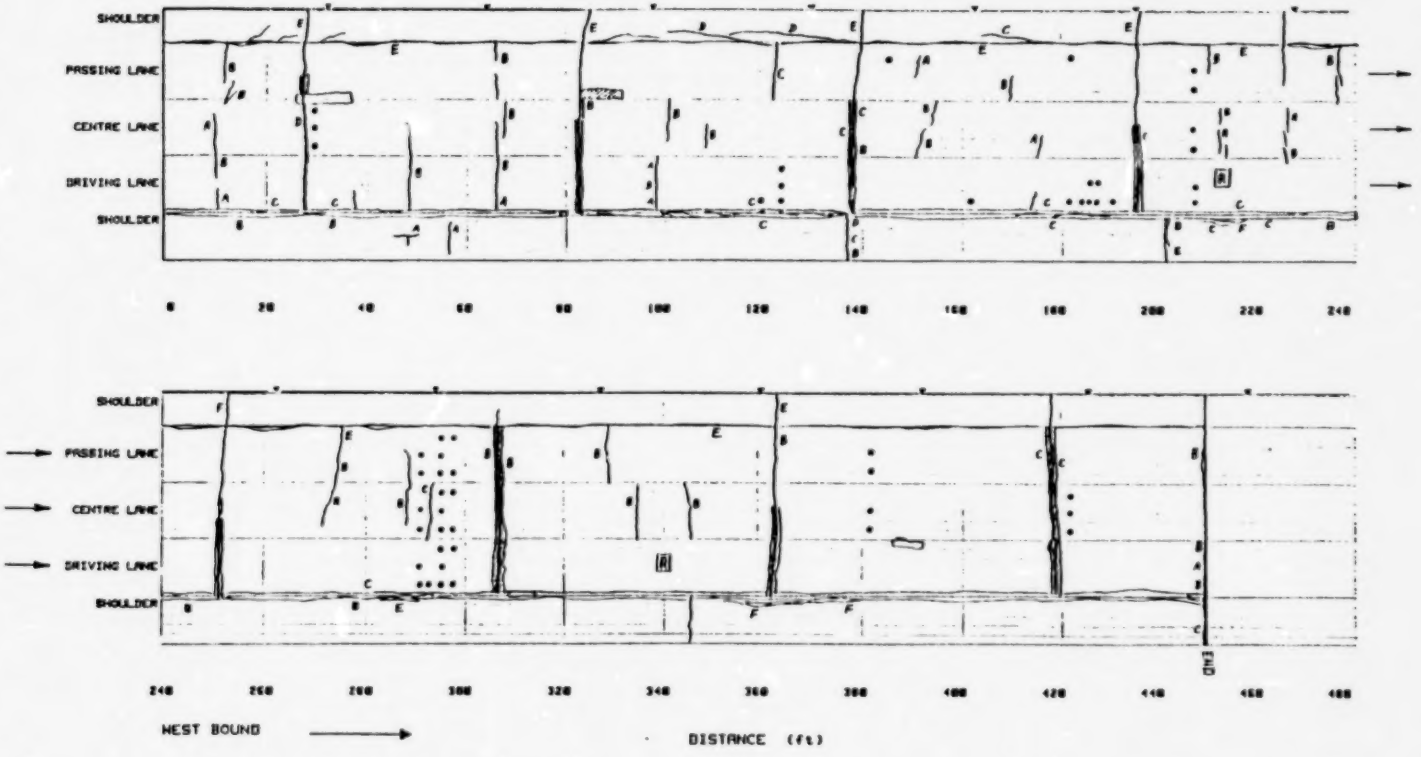


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 10

KEY

- CRACKING
  - Code: A= Hairline
  - B= < 0.2" (< 5mm)
  - C= 0.2-0.4" (5-10mm)
  - D= 0.4-0.6" (10-15mm)
  - E=0.6-0.8" (15-20mm)
  - F= > 0.8" (> 20mm)
  - R= healed over
  - T= treated
- STEPPING
- SETTLEMENT, HEAVING, ETC.
- BLOW-UP
- CONSTRUCTION JOINT
- RAVELLING
- DELAMINATION
  - Code: A= slight
  - B= Medium
  - C= severe
- PATCHING
- OVERLAY
- CORE TAKEN
- RUT DEPTH MEASURED R

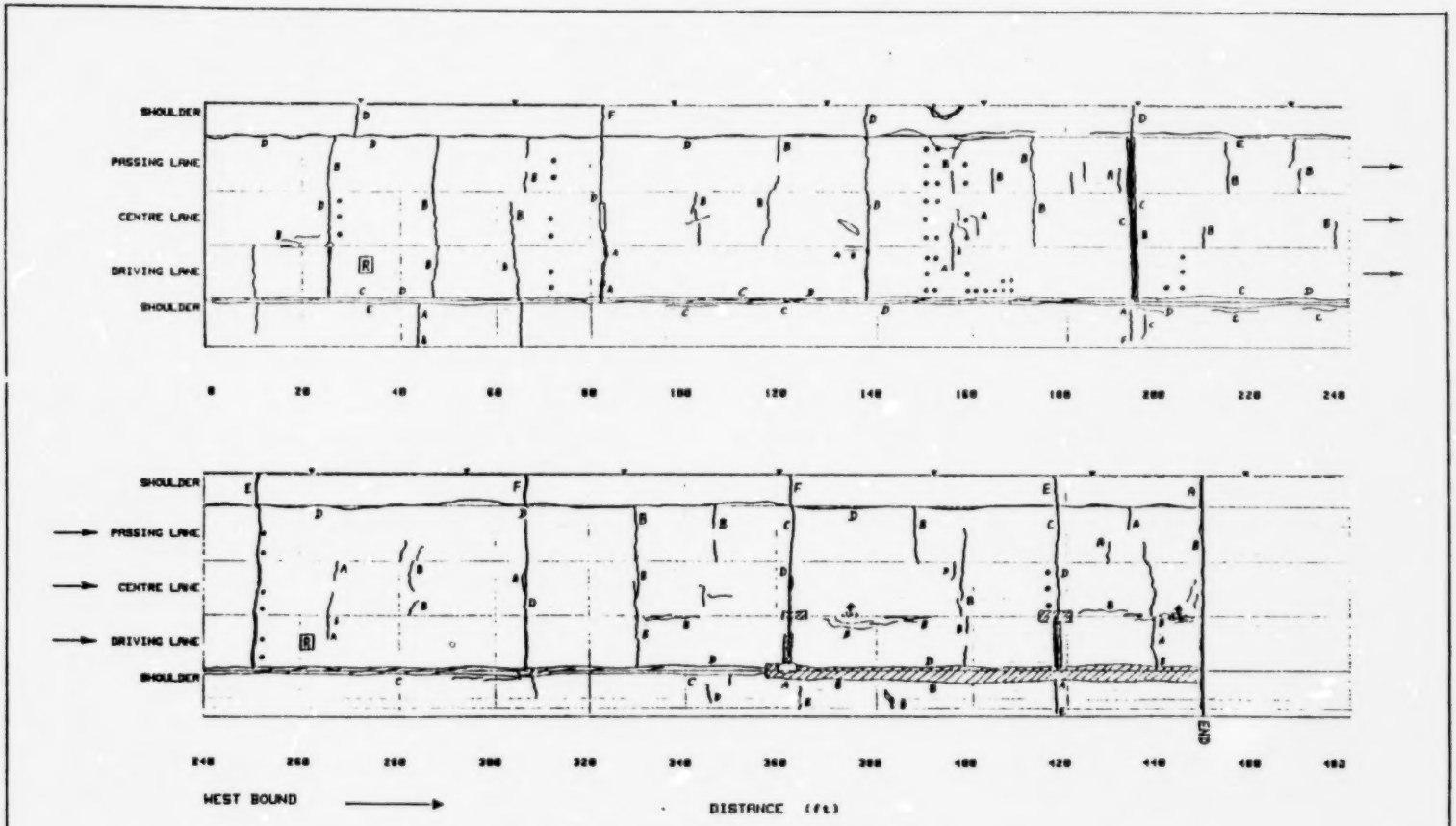


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 11

KEY

* CRACKING
  Code: A= Hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E= 0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  I= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED R

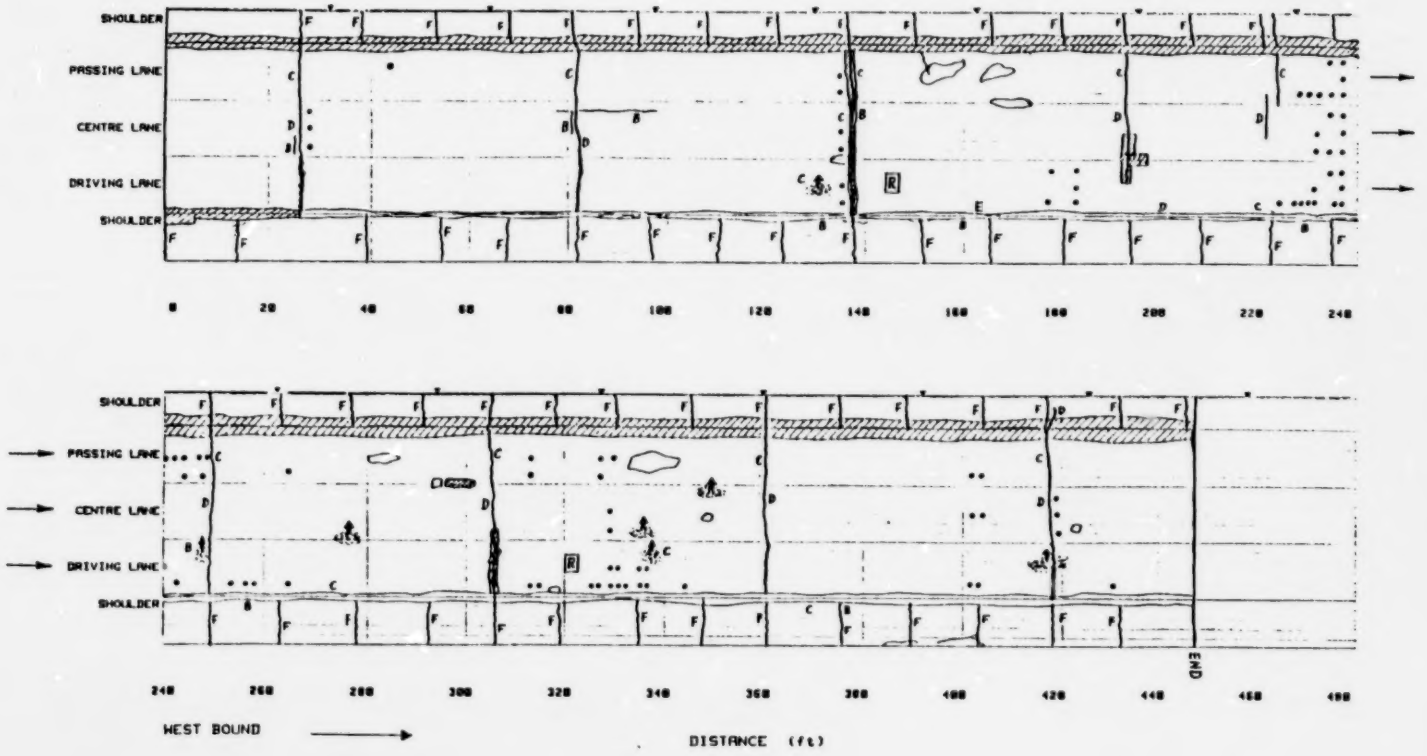


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 12

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2 TO 0.4" (5-10mm)
  D= 0.4 TO 0.6" (10-15mm)
  E=0.6 TO 0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED R

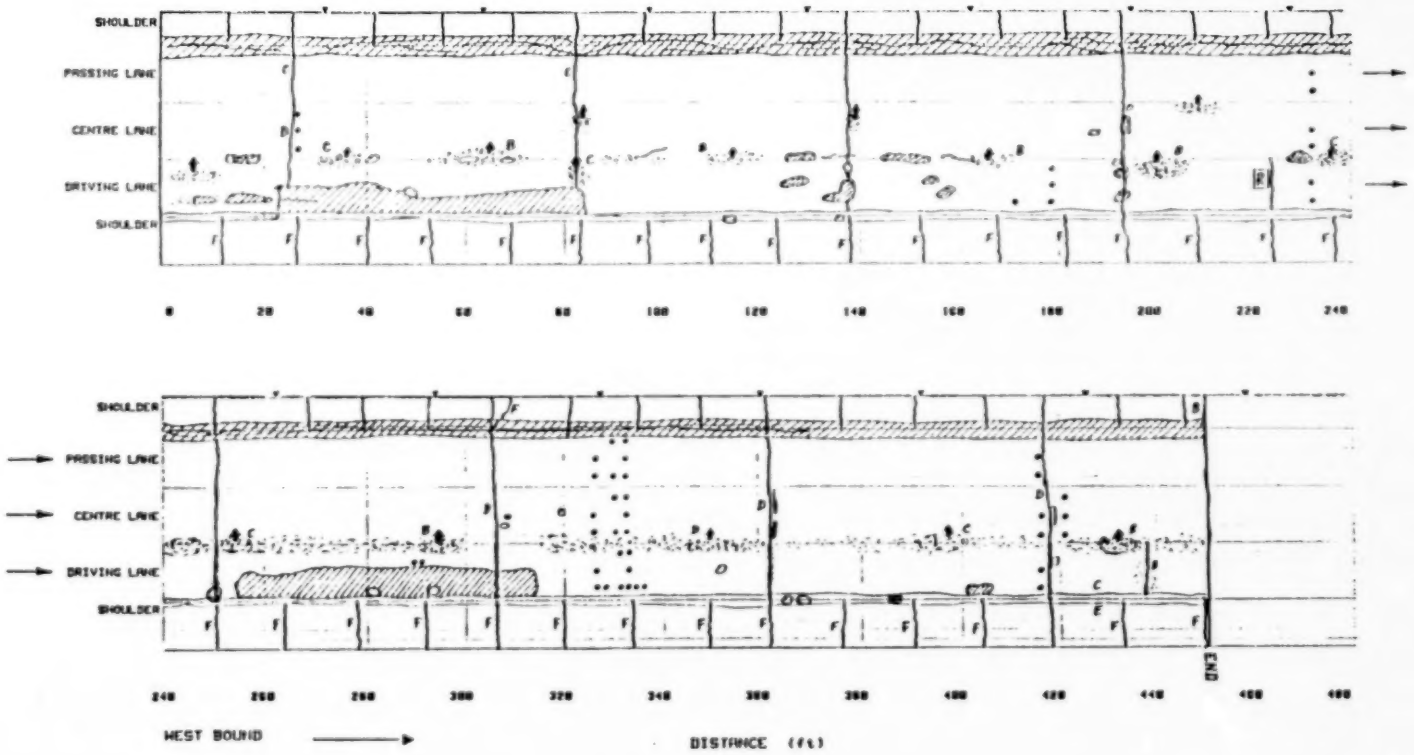


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 13

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED [R]

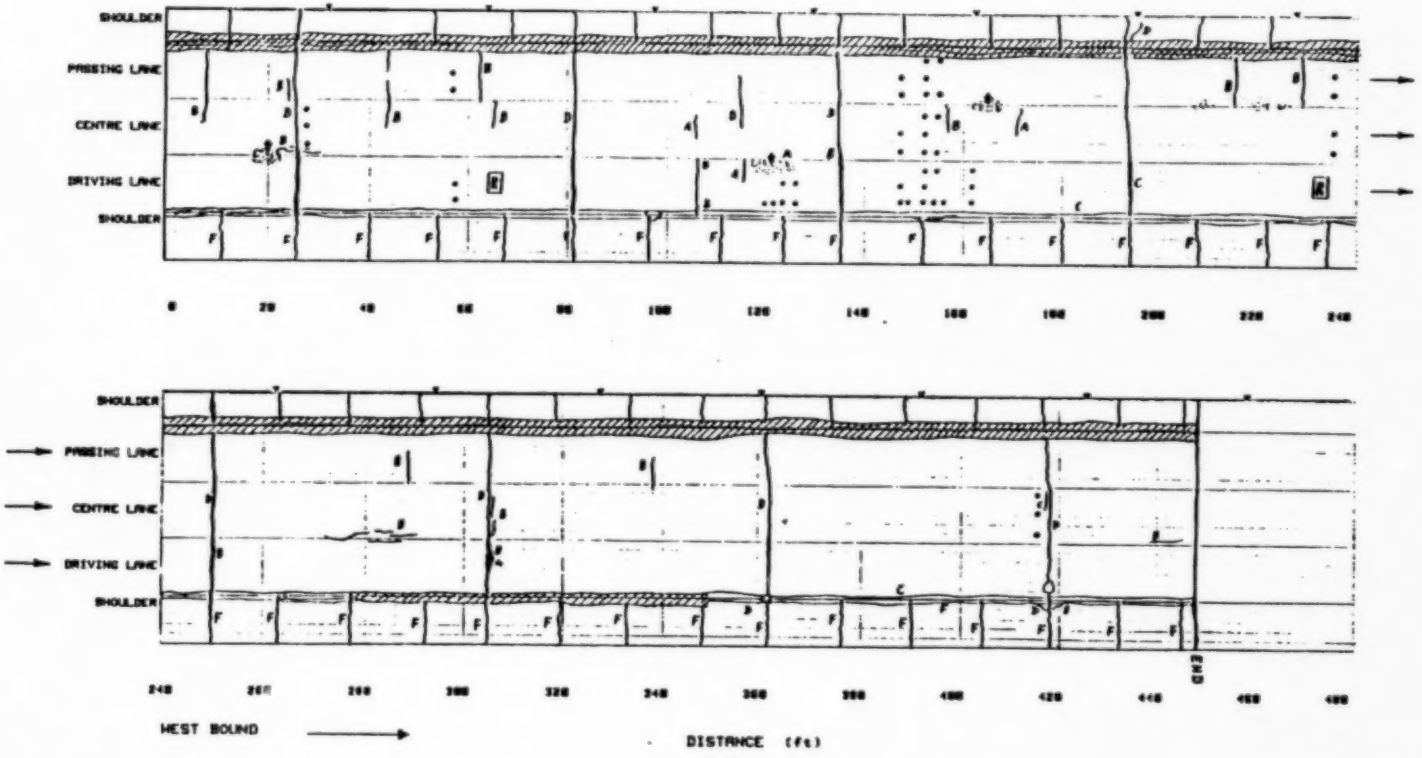


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 14

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* HAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED (1)

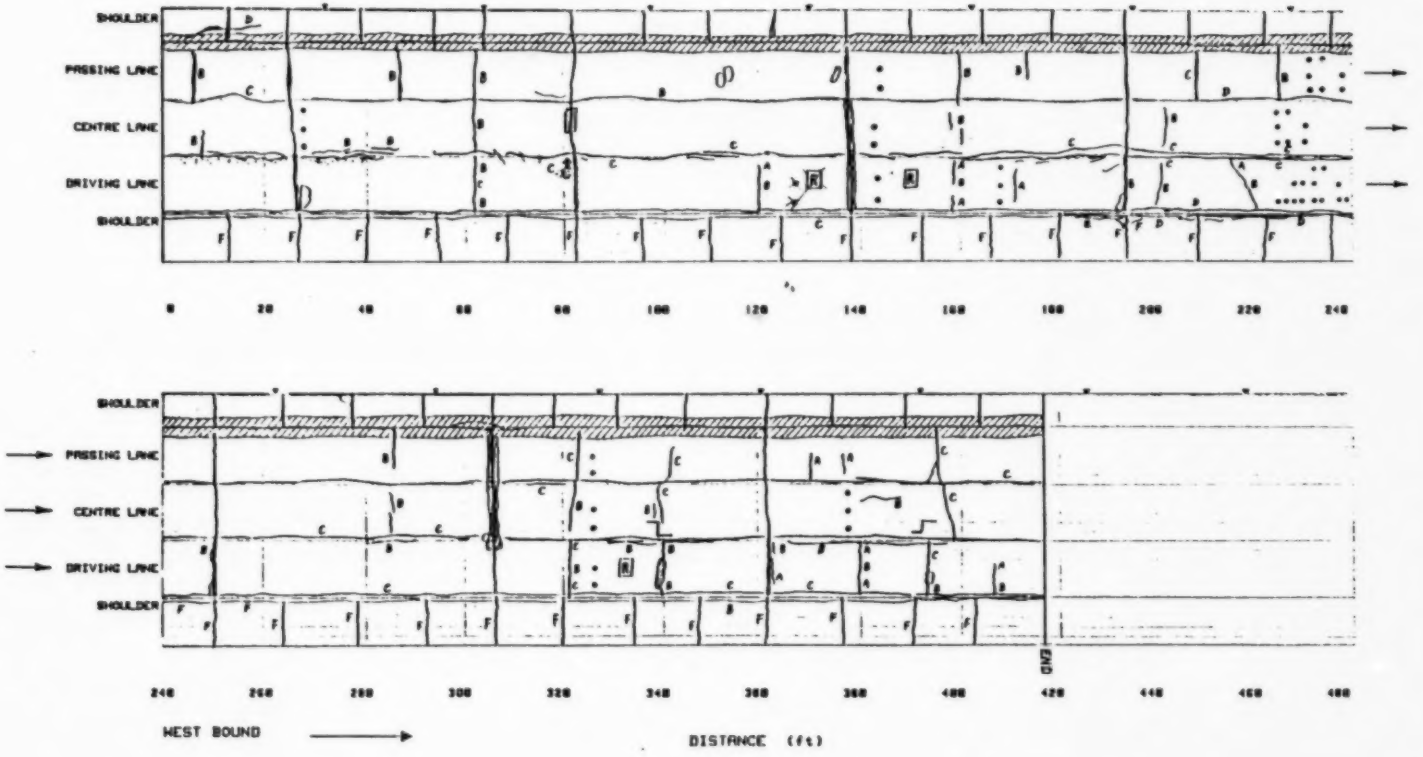


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 15

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED [A]

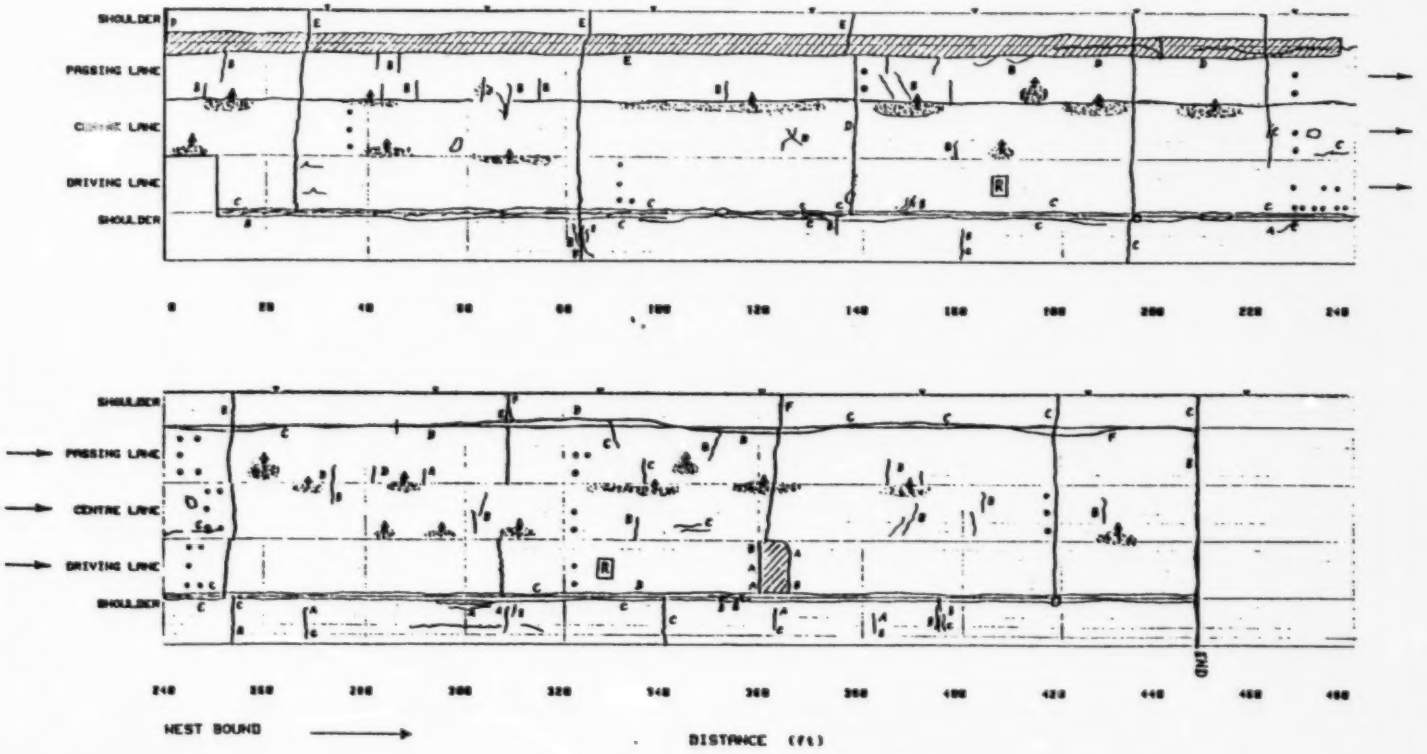


CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 16

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.8" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED R



CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 17

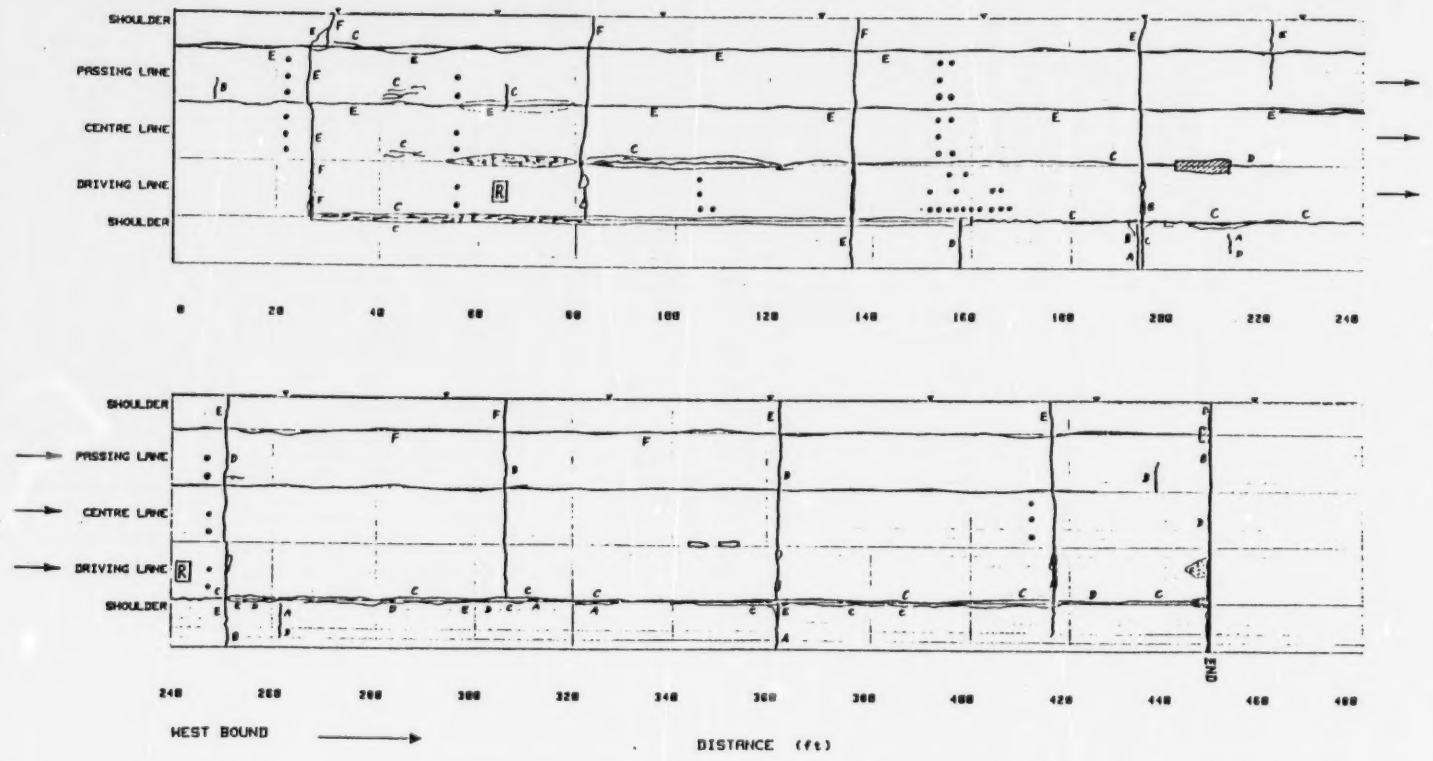
KEY

* CRACKING
Code: A= hairline
B= < 0.2" (< 5mm)
C= 0.2-0.4" (5-10mm)
D= 0.4-0.6" (10-15mm)
E=0.6-0.8" (15-20mm)
F= > 0.8" (> 20mm)
H= healed over
I= treated

* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING

* DELAMINATION
Code: A= slight
B= Medium
C= severe
* PATCHING

* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED [R]



CONDITION SURVEY OF PAVEMENT SURFACE – 11 YRS AFTER CONSTRUCTION

TEST SECTION NO. 18

KEY

* CRACKING
  Code: A= hairline
  B= < 0.2" (< 5mm)
  C= 0.2-0.4" (5-10mm)
  D= 0.4-0.6" (10-15mm)
  E=0.6-0.8" (15-20mm)
  F= > 0.8" (> 20mm)
  H= healed over
  T= treated
* STEPPING
* SETTLEMENT, HEAVING, ETC.
* BLOW-UP
* CONSTRUCTION JOINT
* RAVELLING
* DELAMINATION
  Code: A= slight
  B= Medium
  C= severe
* PATCHING
* OVERLAY
* CORE TAKEN
* RUT DEPTH MEASURED